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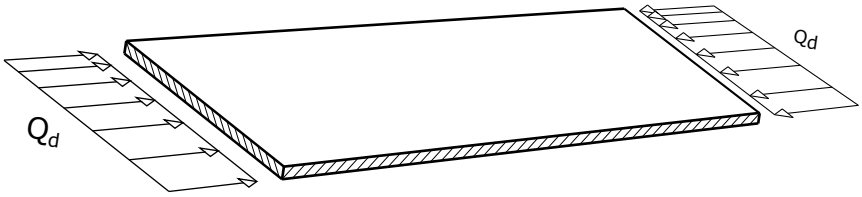

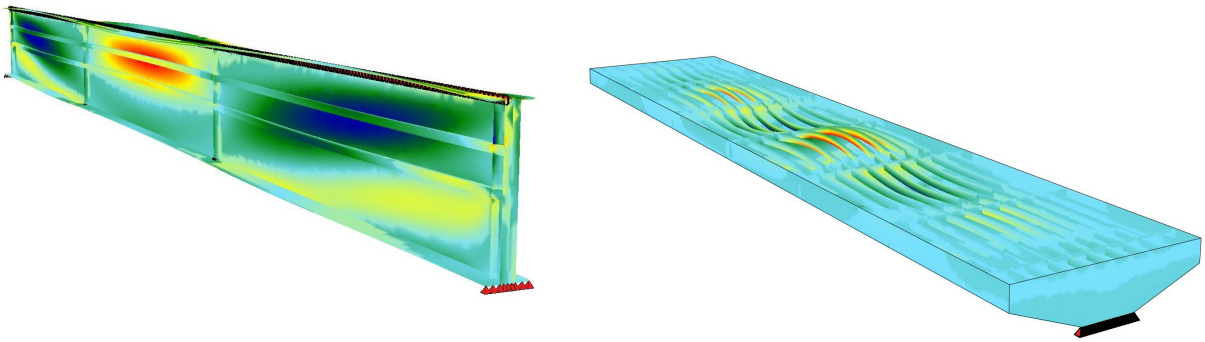

Parametric design of class 4 steel cross sections

A comparison between the reduced stress method and the effective width method

Master's thesis in Master Program Structural engineering and building technology

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Cover: An illustration of a steel bridge.

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Abstract

Eurocode provides two different methods to design steel structures in cross section class four which is the effective width method and the reduced stress method. The first mentioned method is the most common method in Sweden and consists of a rather time consuming calculation procedure. The second method results in a rather conservative design. The aim with this thesis is to achieve a better understanding of when and how to use either of the methods, and to achieve an effective design process with the help of a developed design tool.

The thesis is conducted by a literature study about the two methods and the existing studies of improved versions of the methods. The literature study is followed by a parametric study where the methods are compared. The reduced stress method is both calculated analytically and combined with the FE-programme Sofistik with its geometry generated with Grasshopper, whereas the effective width method consists of hand calculations.

Keywords: structural engineering, bridge engineering, steel cross section class 4, parametric design, reduced stress method, effective width method.

Sammanfattning

Eurocode föreslår två olika beräkningsmetoder för att räkna på tvärsnitts klass fyra stålkonstruktioner. Den första metoden är flitigt använd i Sverige och kallas för den effektiva bredd metoden. Ett Problem med metoden är att den i vissa fall kan bli tidskrävande. Den andra metoden kallas för den reducerade spännings metoden och är känd för att vara den lite mer koncervativa utav de två. Målet med arbetet är att identifiera var och när ena metoden kan vara mer fördelaktig än den andra samt att uppnå en tidseffectiv design process med hjälp av ett genomarbetat design verktyg.

Examensarbetet består av en litteraturstudie för att först och främst erhålla en djup förståelse kring de två undersökta metoderna, men även kunskap om studier som har gjorts kring förbättringar av metoderna. Tre olika konstruktioner är undersökta i det här examensarbetet där konstruktionernas kapacitet med de olika metoderna är jämförda. Parametriska studier har genomförts på ett lådtvärsnitt för att undersöka vilka parametrar som påverkar resultaten och hur metoderna bäst används.

Acknowledgements

First of all we would like to thank our supervisors Victor Andersson and Gustav Good at Ramboll as well as our examiner Mohammad Al-Emrani for their support and guidance throughout this master's thesis project. Their commitment not only to the progress of our work but also our well being has been crucial for us. We would also like to thank our opponents Amer Bitar and Pedro Cobucci for their thoughts and questions as a part of the development of this thesis. Finally we would like to thank the rest of Ramboll and in particular the bridge division at Ramboll for thier support and interest in our work.

This report is performed as a final assignment written during the last six months of our five years long education. We have gotten the possibility to apply our knowledge from previous years at Chalmers to a specific but extensive project which has been challenging but of course very educational. Moreover we have gotten the opportunity to learn more about different programs and calculation methods which we've never used before. We will bring this with us in our future work as engineers. We are proud to get the opportunity to publish this thesis and we hope that the subject will be further investigated in the future.

Malin Stening and Maurice Kronberg, Gothenburg, June 2020

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1
Introduction

When designing a steel structure according to Eurocode, regards needs to be taken for the risk of it's compressed parts to buckle. This is accounted for in Eurocode by a cross section classification with four different classes. Cross section class 4 means that local buckling will occur before the yield stress is reached. The capacity of class 4 cross sections shall consequently be limited with regard to this. [SIS, 2011a] Modern steel bridges are often very slender in order to achieve a slim design and save material. As a result their cross section tends to be in class 4 which reduces their capacities.[Zizza, 2016]

1.1 Background

Eurocode provides two different methods to design steel structures in cross section class 4, "Effective width method"(EWM) and "Reduced stress method"(RSM). [SIS, 2011b]

EWM is a frequently used method to calculate the strength of steel plates in bridge designs. Mostly the method corresponds quite well to the post buckling capacity which is why the method often is preferred [Gren et al., 2019]. Plates subjected to compression has capacity left after the plate starts to buckle. The EWM accounts for this effect by calculating an effective width and introducing the effective cross section.[Hansen et al., 2010] Some parts remains active whereas some parts of the cross section are considered not to withstand load efficiently.[SIS, 2011b]

RSM, which is briefly described in EC3-1-5 Chapter 10, does not consider redistribution of stresses within the element which in some cases under predict the capacity of the cross section which then gets over conservative. On the other hand, calculating the effective cross sections is not needed which is a rather time consuming process. Another benefit compared to EWM is that the method allows for calculating the buckling capacity of multiaxially loaded structures which often is the case for modern bridges with a slender design.[Reis et al., 2019]

A problem is that the national application of Eurocode, [TSFS 2018:57, 2018] chapter 19, does not recommend to use the RSM in Sweden. However, the method is not used to the same extent whereas the EWM is well developed and used in design of steel plates. Despite this, the RSM in combination with a FE-programme and parametric design could lead to an optimized and more efficient design procedure.

1.2 Aim

The aim of this thesis is to investigate how the two calculation methods for calculating steel structures with cross section class 4, differ with regard to capacity and calculation time. How the already existing improvements of the methods influences this and if they are safe and reliable in design. To elaborate further with calculation time and capacity a parametric design tool based on RSM will be created. The idea of the tool is to quickly generate a geometry to implement into a FE-software. The outcome of this thesis should be a better understanding of when and where to use either of the methods and a quick and effective design process with the help of the design tool.

1.3 Objectives

A literature study and a parametric study, both with the focus on comparing the two methods, will be made to investigate differences in capacity and calculation time. To further reduce the calculation time and to optimize the parametric study, a parametric design tool made in grasshopper will be created. During this process a couple of questions shall be answered:

- Is there any difference between the methods? And if so when and where?
- What are the pros and cons of the methods?
- Could capacity be gained by combining RSM with FE?
- Will the parametric tool save time for a designer?

1.4 Limitations

The thesis will focus on steel structures with cross-section class 4 only. The calculations will only consider buckling capacity. The aim of this thesis is to compare but not further improve the methods in Eurocode, or any existing improvements of them in relevant litterateurs. LT-buckling will not be within the scope of this thesis and the FE-analyses will not cover any non-linear behavior.

1.5 Method

The thesis is conducted by a literature study to receive an overview of the current state of the art and help identifying possible problem formulations within the investigated area. Moreover the literature study will be followed up by a parametric study where the two methods will be compared to each other on real case structures to investigate the capacity. The case study will be preformed by Mathcad calculations and FE-analyses which will result in a parametric design tool with the purpose of optimizing the design process. The idea of the design tool is to create a program in the design software Grasshopper that generates a geometry into the FE-software Sofistik where design- and critical stresses are automatically calculated.

2
State of the art

Steel plates in cross section class four can be designed with two suggested design methods, the effective width method and the reduced stress method. Research and numerous experiments have been performed over years which resulted in updated versions of the methods. This chapter will contain background information regarding the two methods, different improvements of the methods and real examples of structures calculated with the RSM.

2.1 Design models

The methods originates from investigations by different scientists, like von Karaman, Winter and von Mises. The following chapter contains background information regarding the two earlier mentioned design models.

2.1.1 Effective width method

The width of the cross section which can resist load efficiently is established according to figure 2.1 in case of uniform compression and internal compression elements [ORI, 2019].

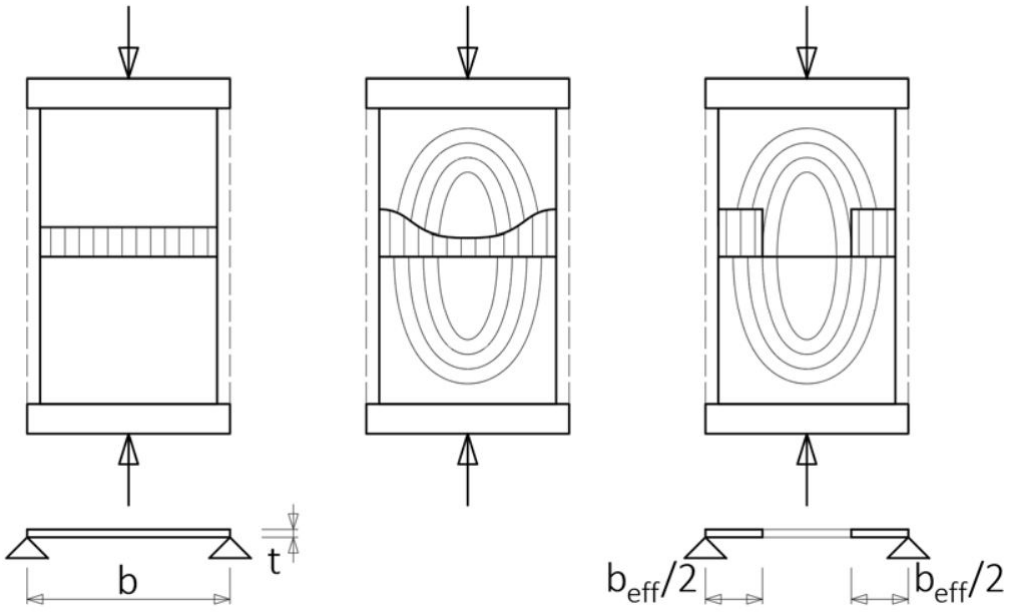


Figure 2.1: Plate subjected to compression [ORI, 2019]

The EWM is based on Von Karaman's investigations in 1932. His findings lead to a formula which enables to calculate an effective width, which does not depend on the total width nor the magnitude of the compressive stress. Karaman concluded that there are stiffer parts along the edges because of redistribution of stresses. The mid part in between the stiff edges can not resist load efficiently in case of plate buckling whereas the stiffer parts can.[ORI, 2019] Several experiments was performed and a modified formula regarding the effective width taking imperfections into account was developed by Winter.[Structures and Ii, 2008] The equation was further modified in the 1968 for cold formed steel which ended up with the same equation as eq. 4.2 in EC3-1-5.

Formula by Von Karaman[Hansen et al., 2010]:

$$b_e = \frac{\pi}{\sqrt{3(1-v^2)}} t \sqrt{\frac{E}{\sigma_e}} \tag{2.1}$$

Formula by Winter[Hansen et al., 2010]:

$$b_e = 1.9t\sqrt{\frac{E}{\sigma_e}}(1 - 0.574\frac{t}{b}\sqrt{\frac{E}{\sigma_e}}) \tag{2.2}$$

The updated version according to EC3-1-5 equation 4.2 [SIS, 2011b] when the plate is subjected to uniform compression is:

$$\frac{b_e}{b} = \sqrt{\frac{\sigma_{cr}}{\sigma_e}}(1 - 0.22\sqrt{\frac{\sigma_{cr}}{\sigma_e}}) \tag{2.3}$$

The calculation process regarding stiffened plates in CSC four starts with the check of the local buckling of the plate fields between stiffeners under the assumption that the stiffeners are stiff enough. Parts of the steel plate are reduced due to buckling which leads to an ineffective part in between.

However, checks needs to be performed regarding the stiffeners since the stiffeners could buckle itself locally or globally. If the stiffeners are not stiff enough, a part of the plate and the stiffener can behave like a column exposed to axial forces with transverse stiffeners as supports. The transverse stiffeners need to be stiff enough to ensure a rigid support to the longitudinal stiffeners. The behaviour can however also be somewhere in between column like or plate like behaviour which is explained in detail in chapter 2.1.3. [SIS, 2011b]

2.1.2 Reduced stress method

The reduced stress method is inspired by the Von Mises yield criterion where the properties from the gross cross section is calculated. A reduced stress limit which is lower than the yield stress is calculated based on the stress distributions of the gross cross-section. Both the properties and the stress limit are calculated separately for every part of the cross section and the weakest part with regard to this will govern the whole cross section. This means that a large difference in capacity between elements will be disadvantageous since no load shedding is accounted for. [Gren et al., 2019]

The method is considered as a conservative method since the weakest section is decisive when calculating the resistance of the whole cross section. The allowable stress in the plate is reduced and a combination formula established from von mises yield criterion enables RSM to check multiaxially loaded structures. The resistance is checked according to the following formulas. [Zizza, 2016]

Cross section class three can be assumed if the condition in EC3-1-5 eq.10.1 is fullfilled regarding multiaxially loaded plates. Unlike the EWM, the slenderness in the RSM is calculated with a global slenderness formula according to [SIS, 2011b] Chapter 10(3);

$$\lambda_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} = \sqrt{\frac{f_y}{\sigma_{eq,cr}}} \tag{2.4}$$

To calculate the global slenderness, $\alpha_{ult,k}$ and α_{cr} needs to be estimated with the following formulas, taken from [SIS, 2011b] Chapter 10(4) and 10(6) respectively. Alternatively using a FE-program to obtain eigenvalues and von mises stresses to calculate $\alpha_{ult,k}$ and α_{cr}.

$$\frac{1}{\alpha_{ult,k}^2} = [\frac{\sigma_{x,Ed}}{f_y}]^2 + [\frac{\sigma_{z,Ed}}{f_y}]^2 - [\frac{\sigma_{x,Ed}}{f_y}][\frac{\sigma_{z,Ed}}{f_y}] + 3[\frac{\tau_{Ed}}{f_y}]^2 \tag{2.5}$$

$$\frac{1}{\alpha_{cr}} = \frac{1+\psi_x}{4\alpha_{cr,x}} + \frac{1+\psi_z}{4\alpha_{cr,z}} + [[\frac{1+\psi_x}{4\alpha_{cr,x}} + \frac{1+\psi_z}{4\alpha_{cr,z}}]^2 + \frac{1-\psi_x}{2\alpha_{cr,x}^2} + \frac{1-\psi_z}{2\alpha_{cr,z}^2} + \frac{1}{\alpha_{cr,\tau}^2}]^{\frac{1}{2}} \tag{2.6}$$

The reduction factor is the smallest value of $\rho_x(\alpha_p)$, $\rho_z(\alpha_p)$ and $\chi_w(\alpha_p)$ calculated with the slenderness, λ_p, according to EC3-1-5 chapter 4.5.4 (1) and chapter 5.2(1). The interpolation between column- and plate buckling to calculate the final reduction factor, ρ_c, is calculated according to chapter 2.1.3. Equation 2.7 is used to verify if the condition is met which leads to the final reduction factor [SIS, 2011b]:

$$[\frac{\sigma_{xEd}}{f_y/\gamma_{M1}}]^2 + [\frac{\sigma_{zEd}}{f_y/\gamma_{M1}}]^2 - [\frac{\sigma_{xEd}}{f_y/\gamma_{M1}}][\frac{\sigma_{zEd}}{f_y/\gamma_{M1}}] + 3[\frac{\tau_{Ed}}{f_y/\gamma_{M1}}]^2 \leq \rho^2 \tag{2.7}$$

An alternative method is to use the following interaction equation (Equation 2.8) instead of the smallest reduction factor. The reduction factor is calculated with the interaction formula in chapter 2.1.3 for each direction.

$$[\frac{\sigma_{xEd}}{\rho_x f_y/\gamma_{M1}}]^2 + [\frac{\sigma_{zEd}}{\rho_z f_y/\gamma_{M1}}]^2 - [\frac{\sigma_{xEd}}{\rho_x f_y/\gamma_{M1}}][\frac{\sigma_{zEd}}{\rho_z f_y/\gamma_{M1}}] + 3[\frac{\tau_{Ed}}{\chi_w f_y/\gamma_{M1}}]^2 \leq 1 \tag{2.8}$$

2.1.2.1 Von Mises yield criterion

Reduced stress method is based on von mises yield criterion[Gren et al., 2019]. It can be concluded that yielding in a material will occur in case von Mises stress exceeds or is the same as the yield limit of a material loaded in tension. The condition can however be investigated experimentally to determine whether the material is yielding.

Von Mises yield criterion [Palaniswamy, 2012]:

$$\sigma = \sqrt{\frac{1}{2}[(\sigma_{xx} - \sigma_{yy})^2 + (\sigma_{yy} - \sigma_{zz})^2 + (\sigma_{zz} - \sigma_{xx})^2 + 6(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2)]} \quad (2.9)$$

The condition regarding von Mises formula leads to a surface or a elliptical curve in case of plane stress(figure 2.2).

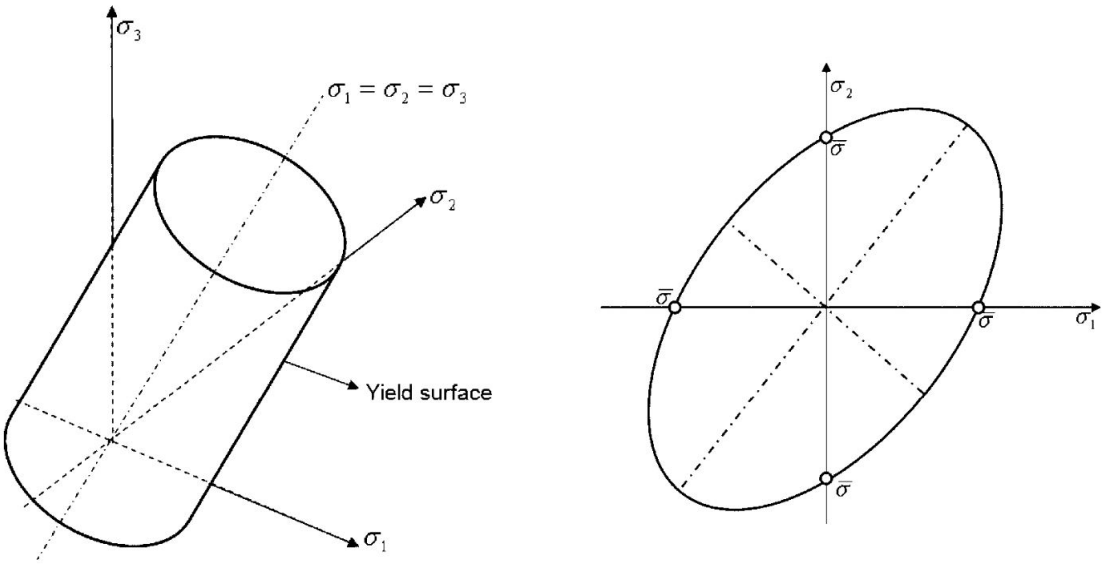


Figure 2.2: Figure of von Mises yield criterion [Palaniswamy, 2012]

2.1.3 Column- versus plate like behaviour

Different behaviour occurs depending on the plates geometry and dimensions. According to [SIS, 2011b], two types of behaviours are the plate buckling and column buckling with an interaction equation in between to achieve a behaviour closer to reality. Sometimes the stiffened plates behaves in a manner in between column and plate like buckling depending on how stiff the plate is(figure 2.3). A large plate with several stiffeners can buckle like a orthotropic plate even though the stiffness in different directions will differ. The interaction equation is however used in the EWM and the RSM in the same way to obtain a final reduction factor. [Matsumoto, 2000]

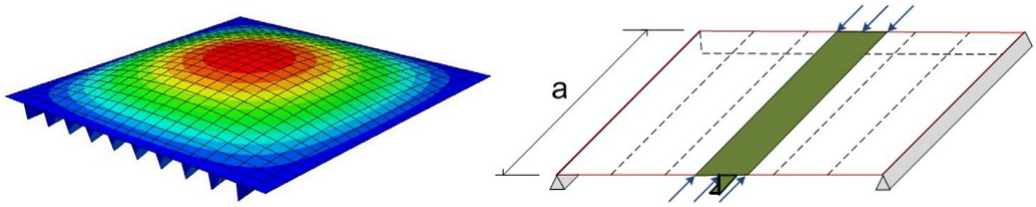


Figure 2.3: Plate like- vs column like behaviour of an stiffened rectangular plate [Matsumoto, 2000]

Regarding calculation of the reduction factor for a plate with plate like behaviour, the critical buckling stress needs to be calculated according to EC3-1-5 annex A.1 which is dependent on the degree of stiffening. A stiffer orthotropic plate can however possibly lead to column like behaviour or affect the reduction factor in the interaction formula in SS-EN 1993-1-5 equation 4.13 (figure 2.4).[Matsumoto, 2000]

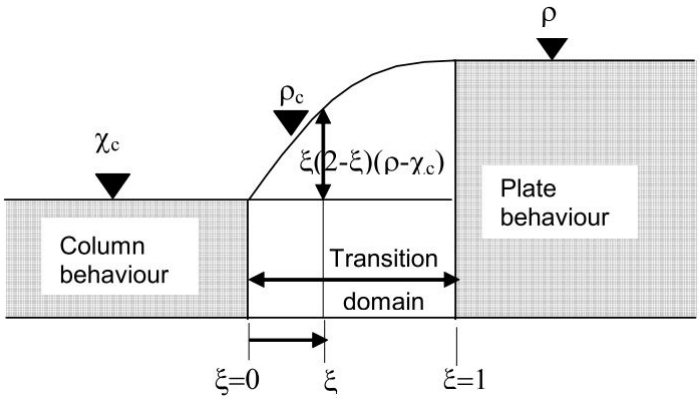


Figure 2.4: Interaction column- and plate like behaviour [Johansson et al., 2007]

The factor ρ is the reduction factor in accordance with the plate like behaviour whereas χ_c is the reduction factor in case of column buckling. The factor ξ depends on the width and length of the plate since the dimensions affects the behaviour of the plate. A length much shorter than the width will lead to a behaviour similar to column buckling and if the width increases the plate will successively behave more in accordance with the plate buckling theory.[SIS, 2011b]

2.2 Scientific studies

Numerous papers has been written with focus on comparing and/or improving the two methods. The common opinion is that RSM is in most cases more conservative and therefore not as material efficient. A general conclusion is also that the EWM is more tedious and not as general as RSM. It's for example not possible in Eurocode to combine multiaxial stresses in one calculation with EWM.[Samvin and Skoglund, 2016] This section will go through interesting findings in relevant literature to get a

better picture of what has been investigated and what conclusions that has been drawn from it. The literature are scientific papers from 2007 to as recent as 2019 which gives a hint on how relevant this subject is, for example scientific papers like [Braun and Kuhlmann, 2012], [Zizza, 2016] and [Reis et al., 2019].

2.2.1 Elaboration with load shedding

EC 1993-1-5 Chapter 10 (1) states that the resistance for buckling of a cross section in CSC 4, calculated with RSM, shall be governed by the weakest element within it. This is conservative since load shedding between elements is disregarded. In 2007 a formulation of different resistance levels where proposed in [Johansson et al., 2007] to achieve a more realistic stress distribution between elements. The resistance level that can be utilized depends on the amount of strain that can be allowed within the weakest element. The resistance levels are listed below and illustrated in a stress strain diagram in Figure 2.5

Level 1 is as the original method in Eurocode where no shedding between elements is allowed so that the buckling resistance depends on the weakest element.

Level 2 allows a limited stress distribution where load shedding up to the capacity of the strongest element is considered.

Level 3 allows to increase the resistance of the cross section up to the yield strain of the weakest element.

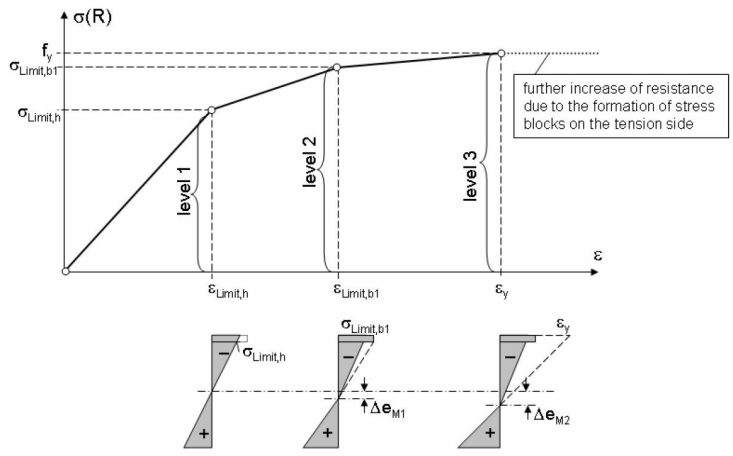


Figure 2.5: Recistance levels according to [Johansson et al., 2007]

A master thesis from 2016 that was carried out at KTH compared EWM and RSM to a non-linear FE analysis [Samvin and Skoglund, 2016]. The RSM calculations where made with all three resistance levels as well as a proposed forth option. This proposed option used interpolation between the stress limits of the weakest and the strongest element to calculate the resistance of the cross section. The comparison was performed by parametric studies of an I-girder and a box-shaped column where

load cases and slenderness alternated.

They concluded that RSM with level 3 resistance as well as the interpolated resistance both gave similar results to EWM for the cases studied in the report. Although problems aroused when axial compression and bending moment where combined, the level 3 resistance calculations for RSM requires an iterative and tedious procedure to calculate the neutral axis for that case. The interpolation formula is based on bending resistance for axial stresses and the cross sectional area for uniform compressive stresses respectively. The interpolation option is therefore not possible to use when bending and axial compression is combined.

Another conclusion, which is quite generally known but nonetheless interesting for this thesis is that it is very suitable to calculate the resistance using reduction factors obtained from a FE-analysis in stead of analytically. The capacities gets closer to reality and the calculations can be preformed fast once the model is built up. The understanding of the designer becomes important as the governing resistance must be interpreted from the analysis which can be hard for an inexperienced designer.

For further work on the subject, the authors suggest a similar investigation but on a whole bridge instead of isolated beams and columns. If a whole bridge is analysed it will be possible to get a better understanding of how much this deviation in capacities between methods can affect for example material use.

2.2.2 Improvement of the interaction formula

A couple of papers has been written regarding the conservative capacity prediction when multiaxial loads are combined according to the interaction formula in Eurocode. Benjamin Brown wrote two articles [Braun, 2010] and [Braun and Kuhlmann, 2012] where a modification of the formula was proposed and compared to existing standards. Antonio Zizza suggested an addition to Brown's modification in [Zizza, 2016] which later on where tested for high strength steel by [Reis et al., 2019]

2.2.2.1 The V factor

Benjamin Braun proposes an alternative way to the verification equation for multiaxial loading in Eurocode regarding the reduced stress method. The proposed change is due to the fact that reduced stress method does not consider redistribution of stresses within elements. The modification should however lead to less conservative results, by adding a V according to the following formula:[Braun, 2010]

$$[\frac{\sigma_{xEd}}{\rho_x f_y/\gamma_{M1}}]^2 + [\frac{\sigma_{zEd}}{\rho_z f_y/\gamma_{M1}}]^2 - V[\frac{\sigma_{xEd}}{\rho_x f_y/\gamma_{M1}}][\frac{\sigma_{zEd}}{\rho_z f_y/\gamma_{M1}}] + 3[\frac{\tau_{Ed}}{\chi_w f_y/\gamma_{M1}}]^2 \leq 1 \quad (2.10)$$

Where
$V = \rho_x \rho_z$ if the stresses, σ_x and σ_z, are in compression. [Braun, 2010]

Another developed concept is the German standard DIN18800-3 (2) which was compared to EN1993-1-5 and equation 2.10. In the German standard, the slenderness and reduction factors are calculated discrete by the stresses from each direction, and the reduction factor depends on the calculated slenderness $\kappa_x(\lambda_{x,p})$, $\kappa_z(\lambda_{z,p})$, $\kappa_{\tau,p}(\lambda_{\tau,p})$. [Braun and Kuhlmann, 2012]

$$[\frac{|\sigma_{x,Ed}|}{\sigma_x, Rd}]^{e_2} + [\frac{|\sigma_{z,Ed}|}{\sigma_{z,Rd}}]^{e_2} - V[\frac{|\sigma_{x,Ed}\sigma_{z,Ed}|}{\sigma_{x,Rd}\sigma_{z,Rd}}] + 3[\frac{\tau_{Ed}}{\tau Rd}]^{e_3} \leq 1 \tag{2.11}$$

Where
$\sigma_{x,Rd} = \kappa_x f_{yd}$
$\sigma_{z,Rd} = \kappa_z f_{yd}$
$\tau_{Rd} = \kappa_\tau f_{yd}$
$e_1 = 1 + \kappa_x^4$
$e_2 = 1 + \kappa_z^4$
$e_3 = 1 + \kappa_x \kappa_z \kappa_\tau^2$
$V = [\kappa_x \kappa_z]^6$ if $\sigma_{x,Ed}$ and $\sigma_{z,Ed}$ are in compression.
$V = \frac{\sigma_{x,Ed}\sigma_{z,Ed}}{|\sigma_{x,Ed}\sigma_{z,Ed}|}$ for other cases.

As can be seen in figure 2.6, the result from the modified formula (equation 2.10) does not differ significantly from the German standard DIN 18800-3 except for very slender plates. Hence, the proposed formula needs to be more conservative in case of slender square plates. If the geometry differ from a square, the modified formula is beneficial since it can account for that effect whereas DIN 18800-3 and EN1993-1-5 can not which leads to conservative and favourable results respectively.[Braun and Kuhlmann, 2012]

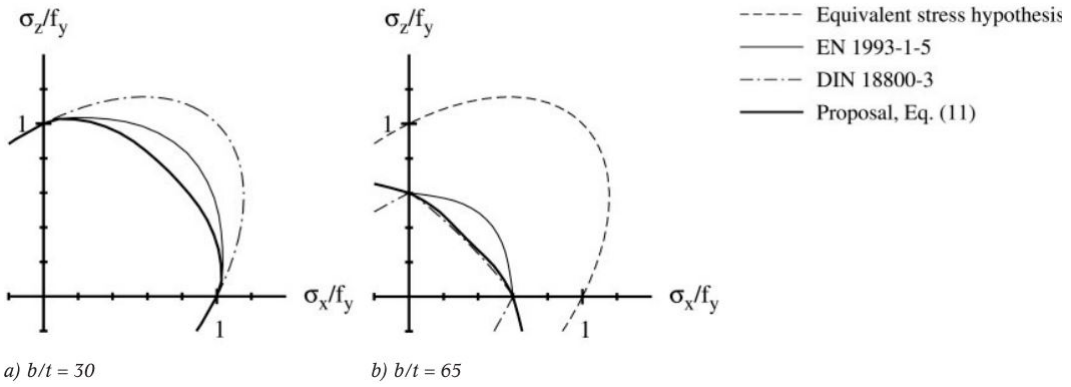


Figure 2.6: Comparison different methods, $\alpha = 1$ [Braun, 2010]

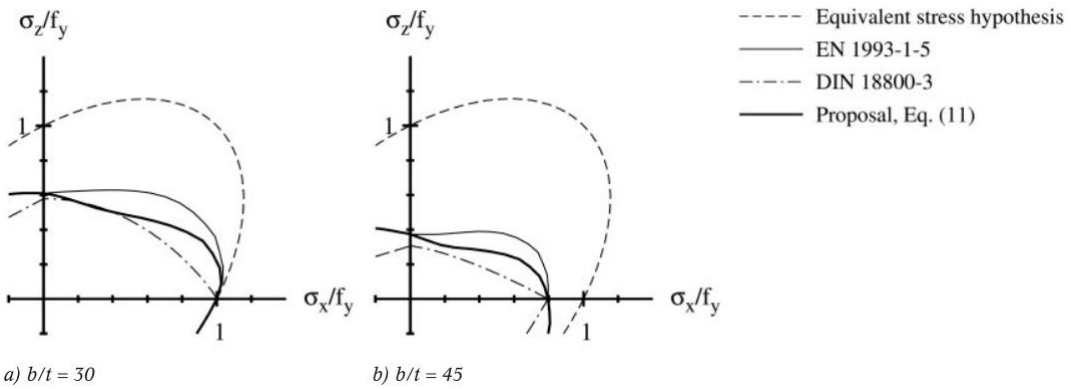


Figure 2.7: Comparison different methods, $\alpha = 3$ [Braun, 2010]

The article concludes that the improved interaction formula entails reasonable results which corresponds quite good to the experiments. Moreover, Benjamin Brown points out that the tensile effect for plates or some part of the plates loaded in tension is not enough studied.[Braun and Kuhlmann, 2012]

2.2.2.2 Addition to the V factor

Antonio Zizza came up with an improved application to Benjamin Brauns formula by introducing a "c", a changed e_3, a square root and another equation for "V" when the plate is subjected to tension. The improved interaction formula was developed from experiments, proposed changes from the past, numerical analysis and comparison between the buckling rules in Germany and the current design rules EC3-1-5. [Zizza, 2016]

$$\sqrt{[\frac{\sigma_x}{\rho_{c,x} f_y}]^2 + [\frac{\sigma_z}{\rho_{c,z} f_y}]^2 - V[\frac{\sigma_x}{\rho_{c,x} f_y}][\frac{\sigma_z}{\rho_{c,z} f_y}] + 3[\frac{\sqrt{3}\tau}{\chi_w f_y}]^{e3}} \leq 1 \tag{2.12}$$

Where
$e_3 = 1,25 + 0,75\chi_w^2$

$V = \rho_{c,x}\rho_{c,z}$ if the stresses, σ_x and σ_z, are in compression. [Braun, 2010]

$V = \frac{1}{\rho_{c,x}\rho_{c,z}^{2-\zeta_x}}$ if at least one of the stresses,σ_x and σ_z, are in tension. [Zizza, 2016]

$\rho_x = c[\frac{1}{\lambda_p} - \frac{0,22}{\lambda_p^2}]$

$c = 1,25 - 0,25\psi \leq 1,25$

$\rho_{c,z} = \chi_{c,z} + [\rho_z - \chi_{c,z}]f$

$f = \lambda_p^{0,5} ln[\xi_z + 1]^{0,9}$

$\xi_z = \frac{\sigma_{cr,p,z}}{\sigma_{cr,c,z}} - 1$

A recently published paper that focuses on design with high strength steel points out the lack of availability to account for the positive effects of tensile stresses in buckling calculations of slender plates subjected to multiaxial stresses.[Reis et al., 2019] This is an important aspect when it comes to material use especially when high strength steel is being used. High strength steel is expensive so the design must be as slim as possible to make it economically advantageous to use it. The paper investigates how well the improved interaction formula from [Zizza, 2016] describes the positive effects of tension stresses for high strength steel. The investigation was conducted by a lab test and a numerical analysis to compare to RSM. It could be concluded from the investigation that it is beneficial to consider the positive effects of tension stresses in the reduction factor. The results from RSM and the numerical analysis agreed well which proves that the improved reduction factor gives a precise and material efficient prediction of the capacity for the studied example.

2.2.3 RSM in structural design

There are two interesting papers that present cases where RSM have been used in design of steel structures. A high velocity railway station roof in Florence [Majowiecki and Pinardi, 2010] and the Queensferry Crossing in Scotland [Gren et al., 2019]. Eurocode did not allow the use of EWM in any of the cases but in the second paper a proposed approach where applied in order to compare the methods.

2.2.3.1 RSM on a high velocity railway station roof

There are cases when EWM is not applicable, for example when the geometry of the structure is complex so that multi-axial stresses occurs. The paper [Majowiecki and Pinardi, 2010] mentions a real case where RSM has been used to verify the design of a railway roof. The roof is located in Florence and has the shape of a cylindrical vault which is formed with a grid of romboids as in figure 2.8 below. The roof cantilevers at the openings and the usage of slender plates imposes reduction of the capacity with regard to buckling which made RSM suitable for the verification.

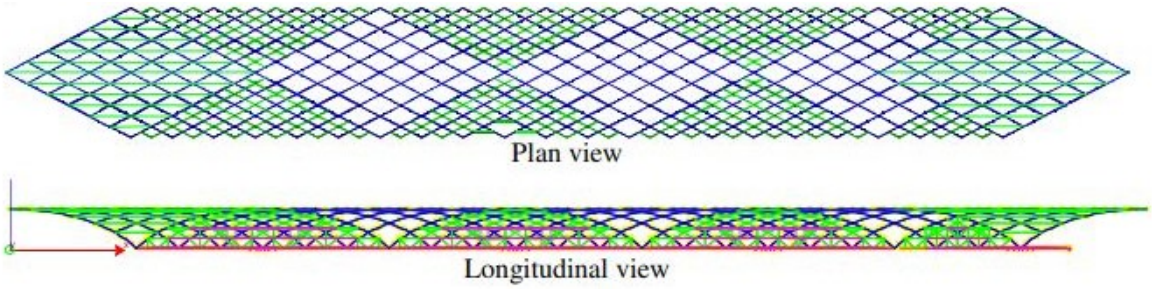


Figure 2.8: Arch roof, high velocity railway station [Majowiecki and Pinardi, 2010]

2.2.3.2 A combination formula for EWM

A recently published paper from "The Nordic Steel Construction Conference" in Copenhagen suggests a combination formula to combine transverse and longitudinal stresses for EWM. [Gren et al., 2019] The formula is presented below in equation (2.13). It is very similar to the one for RSM but with some minor changes.

$$\sqrt{[\frac{\sigma_x}{\rho_x f_y/\gamma_{M1}}]^2 + [\frac{\sigma_z}{\rho_z f_y/\gamma_{M1}}]^2} + 3[\frac{\tau}{\chi_w f_y/\gamma_{M1}}]^2 \leq 1 \tag{2.13}$$

The paper is written by Ramboll employees in an attempt to optimize the design of the Queensferry Crossing which is a stay cable bridge with an orthotropic composite deck over three internal supports. The location of the stay cable planes creates transverse stresses in the steel part of the bridge. The transverse stresses in combination with the global longitudinal stresses "forced" the designers to use RSM as it's not possible to preform multiaxial calculations in eurocode with EWM. A bottom plate from the used design was compared to the same calculated by EWM and a non-linear FE-analysis. Their findings where that EWM with the combination formula describes a correct physical behaviour and generally leads to a higher capacity compared to RSM. This combination formula is not yet tested for other cases nor approved in design but if that will be the case then the attraction for EWM will increase. It should however be pointed out that the earlier mentioned improvements on the combination formula from [Braun and Kuhlmann, 2012] and [Zizza, 2016] for RSM is not used in this investigation since that's not added into Eurocode either.

2.2.4 Conclusions from the literature study

As of now it is not possible to account for load shedding between plates, stress distribution within plates and the positive effects of tensile stresses in multiaxialy loaded plates with RSM in Eurocode. This makes the method over conservative compared to EWM but as these papers has shown there are suggested improvements to account for these effects which can make it more appealing and material effective to use. The method is known to be less tedious, more general and suitable to use in combination with FE-analyses so it would be advantageous if it's precision could match EWM. It would be interesting to compare the EWM with the combination formula proposed in [Gren et al., 2019] to the RSM with the modification to the interaction formula in [Zizza, 2016]. This should preferably be done on a whole bridge structure where some plates are subjected to multi-axial stresses where one is in tension.

3
Single plate

3.1 Method

To justify the calculation procedures on the more complex geometries a single plate is first studied. The plate is loaded in pure compression on the short edges so the two methods should give the exact same result for this case. The measurements of the plate and the loading is presented in figure 3.1 and table 3.1 below.

3.1.1 Description

The single plate is three metres long and two metres wide with a thickness of 20 millimetres. It is loaded with a uniformly distributed load(Q_d) on the short edges on both sides so that a continuous stress field along the length of the plate will occur. Because of the dimensions and since there is no stiffeners there will only be plate like buckling. A figure sketch of the plate along with it's geometrical- and material properties is presented below in figur 3.1 and table 3.1.

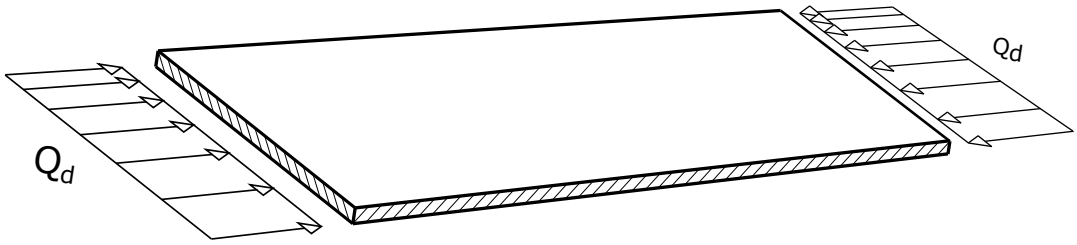


Figure 3.1: Figure of the studied plate

L	b	t	f_y	E	ν	γ_{M0}	γ_{M1}
3 m	2 m	20 mm	235 MPa	210 GPa	0.3	1	1

Table 3.1: Measurement of the plate

The calculation procedure for the two methods looks a little different but some parts coincide. Critical stresses are used to some extent in both methods and the reduction factors uses the same equations but with different slenderness approaches. An attempt to express the two procedures with a flow chart is presented in figure 3.2 below. Note that this is the procedure for this particular example, the general idea is always the same but some parts gets a little more complicated with more complex

structures. An example is that the first assumption in the reduced stress method calculation is that the transverse stresses and the shear stresses are set to 0. If that's not the case, then these stresses need to be interpreted in the amplification factors along with the longitudinal stress, a formula for this is presented in chapter 10 in [SIS, 2011b]. The parts that consider column-like behaviour is unnecessary for this plate since it won't behave like that but the calculations are needed to be able to conclude that that's the case.

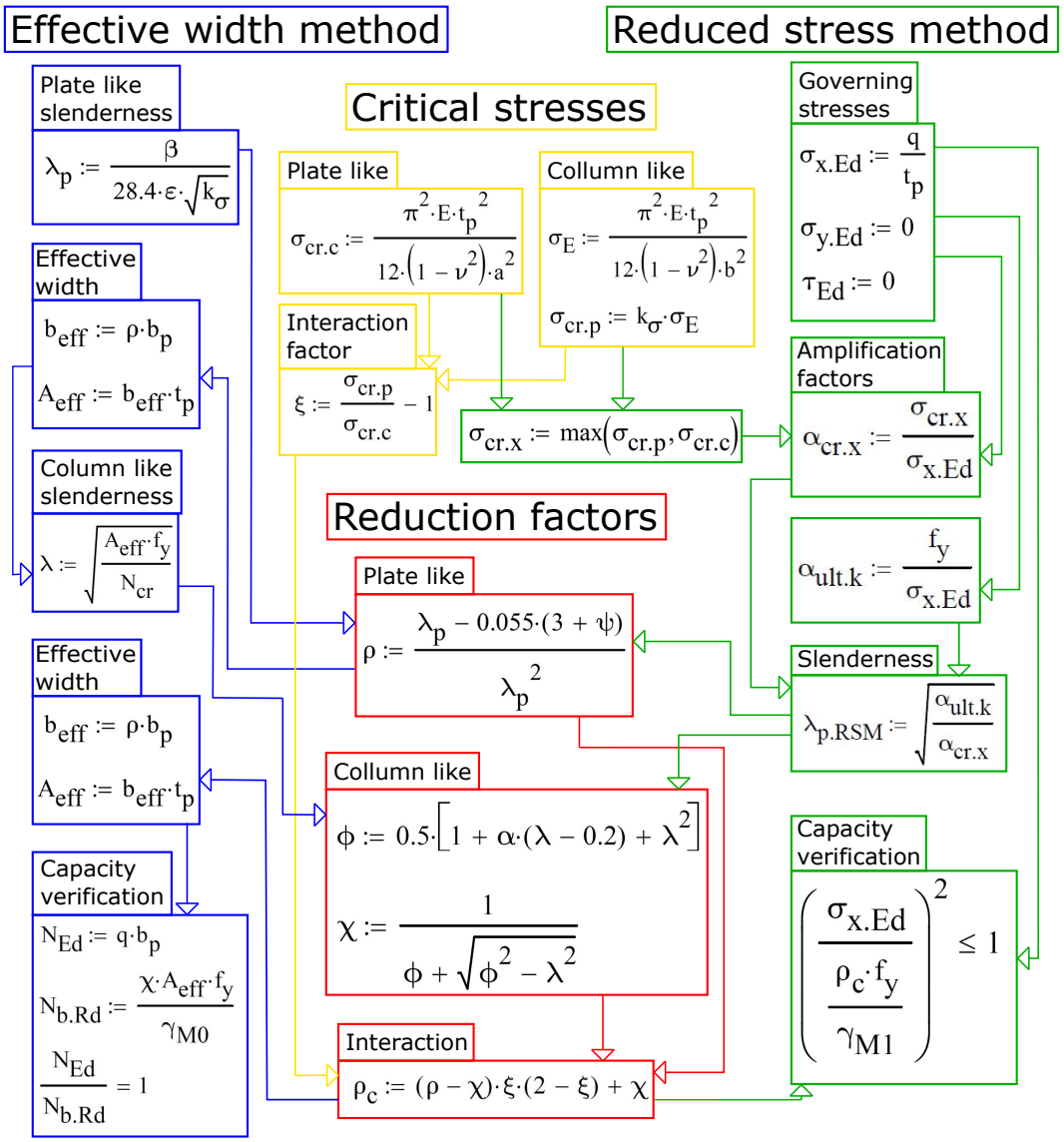


Figure 3.2: Flow shart of the calculation procedure with both methods

The idea with the flow chart is to follow the blue segment for the Effective width method and the green for the Reduced stress method. The yellow and red parts are for critical stresses and reduction factors respectively, these parts are relevant for both methods why they need to be considered separately for both.

Calculating the effective width is a bit iterative when column like behaviour is considered. An effective width considering plate like behaviour is needed to calculate the slenderness for column like behaviour. Both the plate like- and column like reduction factors are then used in the interaction formula where a new reduction factor is calculated and used to calculate the final effective width. This is why the effective width is calculated twice in the flow chart.

3.2 Result

The results are for both methods obtained from analytical calculations of the original plate according to the procedure presented in section 3.1. The resulting capacity is the ultimate capacity, in other words, the load that corresponds to at least 99% utilization of the plate.

3.2.1 Capacity comparison

The capacity comparison between the two methods resulted in almost exactly the same capacity with a difference of about 0.04 % which is a neglectful difference. Notable is that the slenderness's also are close with a difference of 0.03 %. Yet the slenderness in the EWM is based on geometrical- and material properties of the plate while the RSM bases it's slenderness on the two amplification factors which in turn are based on the governing- and critical stresses. The slenderness is used to calculate the reduction factor in both methods but the reduction factors are interpreted diferently in the two methods. The reduction factor in the EWM is multiplied with the width to obtain an "effective width" which then determines the normal force capacity. The reduction factor in RSM on the other hand is used directly into the final verification calculation along with the governing stress to obtain a "reduced stress". The results from both methods are presented in table 3.2 below.

EWM	
λ_p	1.76
ρ_p	0.497
$\sigma_{cr.c}$	8.4356 MPa
$\sigma_{cr.p}$	75.92 MPa
ξ	>1
b_eff	0.994
$N_b.Rd$	4672 kN
q_{Ed}	2336 kN/m

RSM	
$\sigma_{x.Ed}$	116.85 MPa
$\sigma_{y.Ed}$	0 MPa
τ_{Ed}	0 MPa
α_{cr}	0.6497
$\alpha_{ult.k}$	2.011
λ_p	1.7594
ρ_p	0.4973
q_{Ed}	2337 kN/m

Table 3.2: Results from analysis of the single plate with the EWM and the RSM.

3.3 Discussion

The methods uses different approaches to calculate slenderness and capacity, yet they obtain almost the exact same result for both. There is a reason for this, the methods are mathematically the same for this case. Some criterion's needs to be fulfilled for this to occur which is the case in the analyzed plate. The structure should only be conducted by one plate, stresses should only occur in one direction and normal steel material properties should be used. When this occurs the expressions for calculating slenderness and capacity contains the same variables and constants that adds up to the same value. This wont be the case in the structures analyzed in the following sections. The reason for the difference, which is very small but yet a difference, is most likely because of rounding within the constants in the methods or in our calculations.

4

I-Beam with two longitudinal stiffeners

4.1 Method

To investigate a more complex structure with the two methods, an I-beam is studied. The studied I-beam is an existing bridge girder which has already been checked for its buckling capacity by Ramboll by using the EWM. The capacities from the existing calculations will be compared with the results calculated with the RSM. The purpose is to investigate how the capacities differs with the EWM and the RSM. Since the beam will not only contain stresses in one direction, the capacities with the two methods are expected to not give exactly the same results.

4.1.1 Description

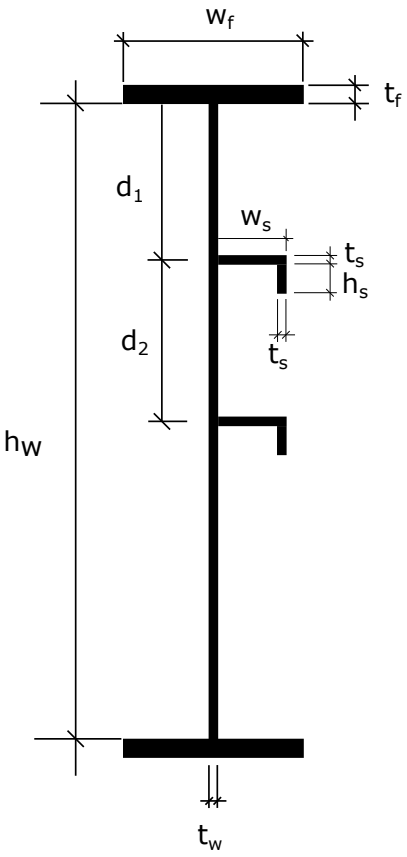
The thin-webbed beam has symmetrical flanges and two longitudinal stiffeners in the compression zone on one side as shown in figure 4.1a. The beam is actually part of a bridge girder with vertical stiffeners at a distance of 8.914 m but here simplified as a simply supported beam with four vertical stiffeners at distance 8.914m with stiff ends which can be seen in figure 4.1b and 4.1c. The comparison is conducted by applying an evenly distributed load Q_D which corresponds to the utilization ratio one, i.e. maximum capacity with regard to buckling. The material properties of the different elements are presented in table 4.1 and the measurements of the beam in table 4.2 along with figure 4.1a.

Part	$\mathbf{f_y}$ **[MPa]**	**E [GPa]**
Web and Stiffeners	390	210
Flanges	380	210

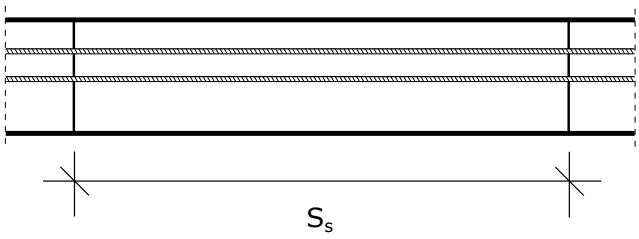
Table 4.1: Material properties of the beam

Measurement	w_f	t_f	h_w	t_w	w_s	h_s	t_s	d_1	d_2	s_s
Length [mm]	560	25	2000	12	200	90	12	500	500	8914

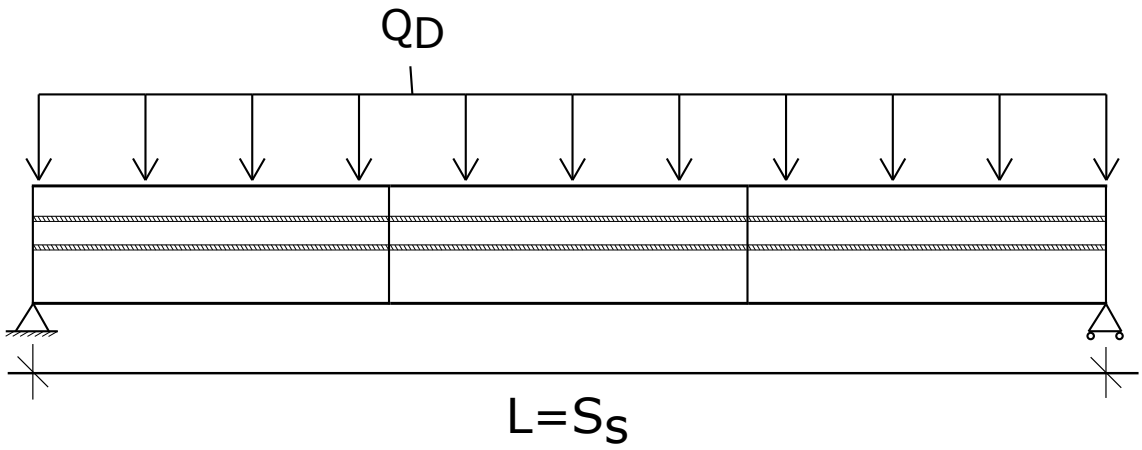
Table 4.2: Measurement of the beam



(a) Cross section of I-beam



(b) Section of I-beam



(c) Simplification of I-beam

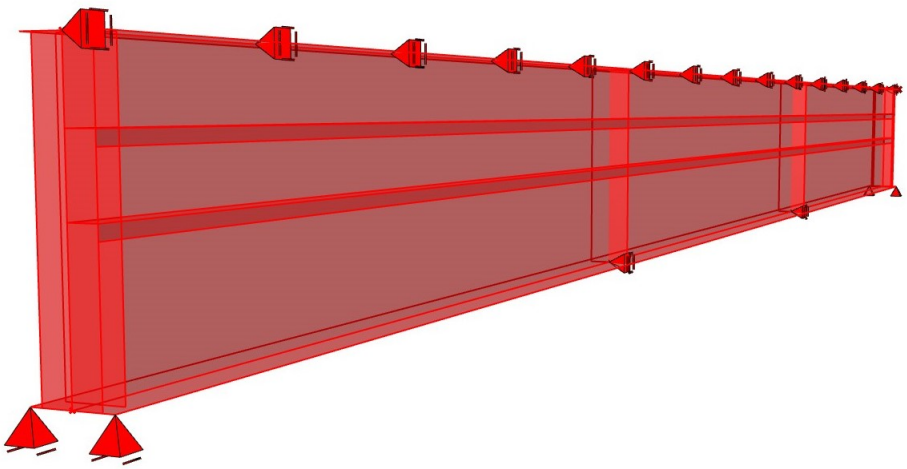
Figure 4.1: Cross section and section of the I-beam

4.1.2 FE-Model

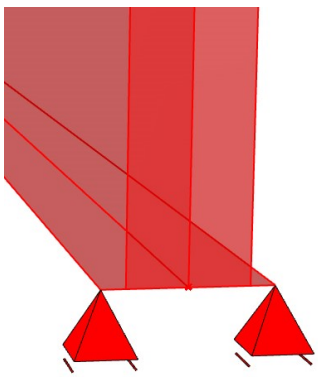
The FE-model is built up as a 3D structure with both triangular and quadrilateral shell elements to be able to catch the whole stress flows of the different plates. The FE-model is presented along with boundary conditions in figure 4.2. The beam is fixed for translation in y- and z-directions along the line at the left support (Se figure 4.2b) whereas the right side is fixed in all directions (Se figure 4.2c).

The top flange is fixed in y-direction along the whole beam length to prevent instability of the beam. The actual girder would be prevented to move in that direction by the bridge deck which makes it a fixation close to reality. At the lower part of the transverse stiffeners, the beam is fixed in y-direction as the actual bridge would be stayed by bracing (Se figure 4.2d). The longitudinal stiffeners and the rest of the transverse stiffeners are not fixed in any direction but merely prevented by their own moment of inertia as well as connection to other elements.

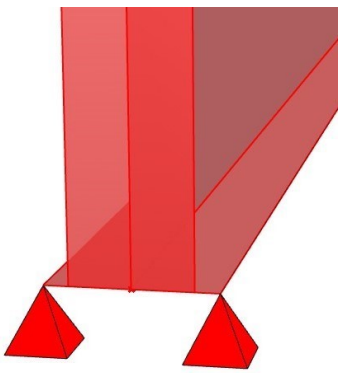
The beam is subjected to an evenly distributed load along the length that is placed at the web center on top of the top flange. The centred placement of the load is in this case not really necessary because of the fixation of the top flange but would in other cases prevent instability problems.



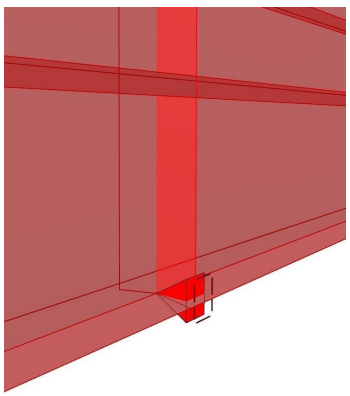
(a) FE model of I-beam



(b) Left BC



(c) Right BC



(d) Mid BC at transverse stiffeners

Figure 4.2: FE-Model

4.1.3 Convergency study

In order to get satisfying accuracy in the calculated results, a mesh convergence study is performed. The maximum deflection in the middle of the beam from an arbitrary uniform load is measured for different mesh sizes and a plot of the variation is presented in figure 4.3 below.

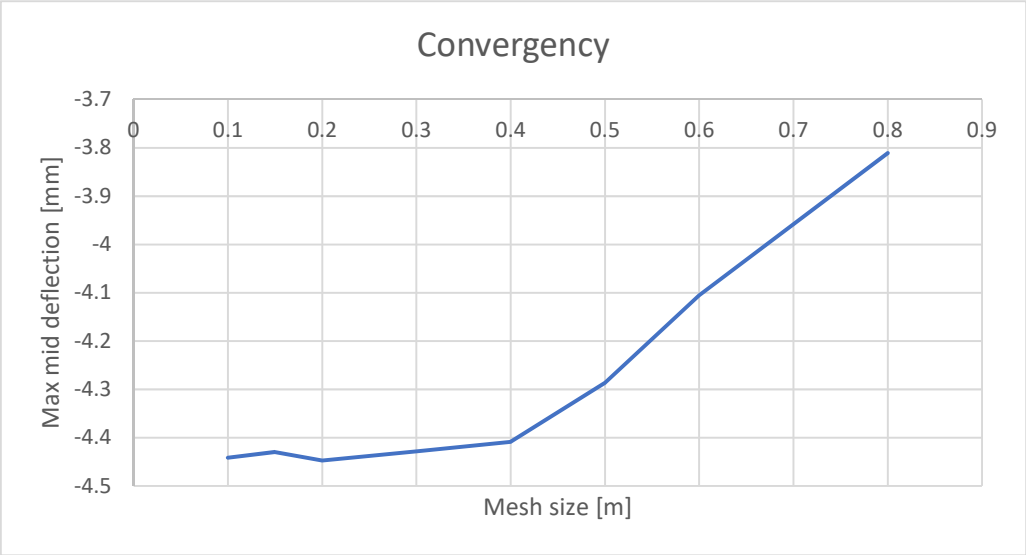


Figure 4.3: Maximum mid deflection of the beam versus mesh size

A maximum tolerance of 1 % difference in deflection between mesh sizes is set as a limit for convergence. What can be seen is that the deflection does not vary as much with meshes finer than 0.4m and the difference for finer meshes is below 1% why a mesh of 0.4m is chosen for the analyses.

4.2 Result & Discussion

The results are based on entirely analytical calculations for the EWM where both moment capacity as well ass shear capacity are accounted for. Interaction between moment- and shear capacity is also calculated with EWM. All according to chapter 4-7 in [SIS, 2011b]. The results for RSM is based on FE- calculations where a linear stress analysis and an eigenvalue buckling analysis are preformed and then accounted for in the hand calculations according to chapter 10 in [SIS, 2011b].

To investigate the I-beam further, it is of interest to study how the capacity is influenced by the type of behaviour of the I-beam. If there is any relationship between the capacity of the beam and the type of behaviour, i.e column like versus plate like behaviour. For this purpose the two methods are calculated and compared with different widths on the longitudinal stiffeners. The idea is to achieve more or

less support for the web to make it behave more or less as a plate. The results are presented in section 4.2.1 and 4.2.2

4.2.1 Capacity comparison

The two methods are quite close to each other with the biggest difference at about 8% if interaction between moment- and shear capacity is considered in EWM. If instead moment- and shear capacity are to be calculated separately in EWM the results will differ more for some stiffener widths. The moment capacity is higher by a lot in all cases so shear capacity will be governing when separated from moment capacity. The results are ploted in figure 4.4 below.

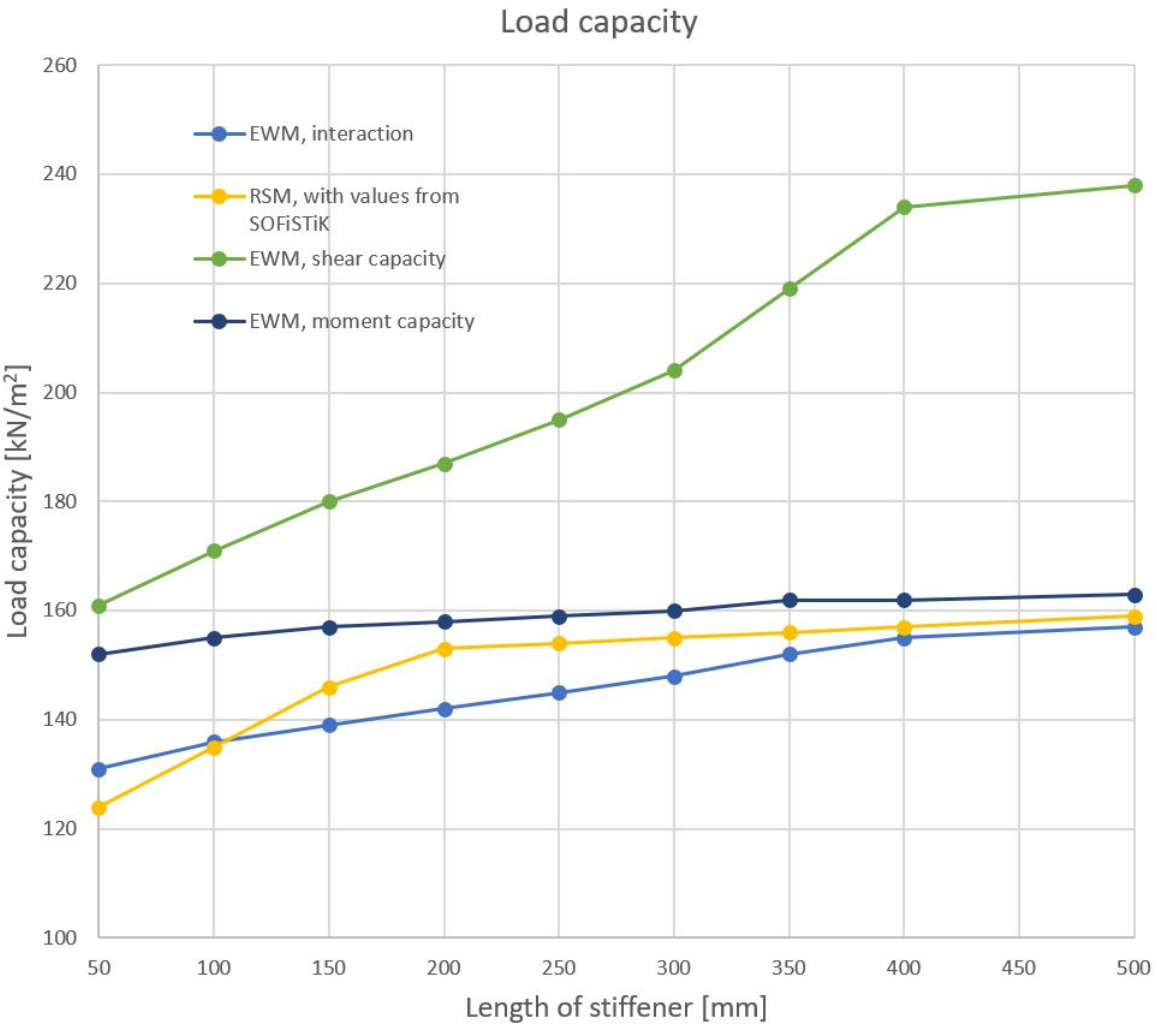


Figure 4.4: Load capacity of the I-beam with EWM and RSM

The capacity increases with wider stiffeners regardless of method which is reasonable since the beam, but mainly the web gains stiffness and support. What is more interesting is that the moment capacity in EWM is barley changed while the shear capacity increases a lot. The initial guess would be the other way around since the stiffeners should increase the webs resistance to normal stress buckling rater than

shear buckling. The answer to this irregular behaviour is that the beam is, even for the most narrow stiffeners, sufficiently supported in terms of pure moment capacity. So an increase in stiffness from the stiffeners will not increase the moment capacity substantially. The behaviour that is visualized in the figure is that the stiffeners becomes more and more rigid so that most of the beam action occurs below the lower stiffener. In a way the beam behaves more and more as an unstiffened, half as high beam. The bottom stiffener, and possibly some part of the web above it, acts as the top flange. A verry stiff top flange indeed, as all the part above it is practically unbendable. The beam stays more or less the same in the context of moment capacity but is now increasingly robust against shear as the beam approaches the smaller beam behaviour. The smaller beam doubles in height to width ratio which explains this high increase of shear capacity. At a stiffener width of 400 mm, it seems that the behaviour is totally in accordance with the small beam and not much more shear capacity is left to be gained.

When the RSM curve is compared to the Interaction curve of EWM they are furthest away from each other in the middle of the section where the beam changes behaviour as previously explained. Almost as if one of the methods fail to capture the transition between the original beam and the half as high one properly. If so, could it be EWM that is unnecessarily conservative or RSM that is over predicting? That would indeed be interesting to test in a lab on actual stiffened beams. The answer might as well lie in the fact that the interaction calculation is a little bit over conservative since the maximum shear and moment wont occur at the same section of the beam.

Another finding is that the RSM curve seems to follow the inclination of the shear curve at the lower stiffener widths and the moment curve with the wider stiffeners. which is further investigated by comparing slenderness in figure 4.5 below. The RSM uses a global slenderness for all elements regardless of buckling behaviour wheras the EWM uses a different slenderness equation depending on the behaviour and element.

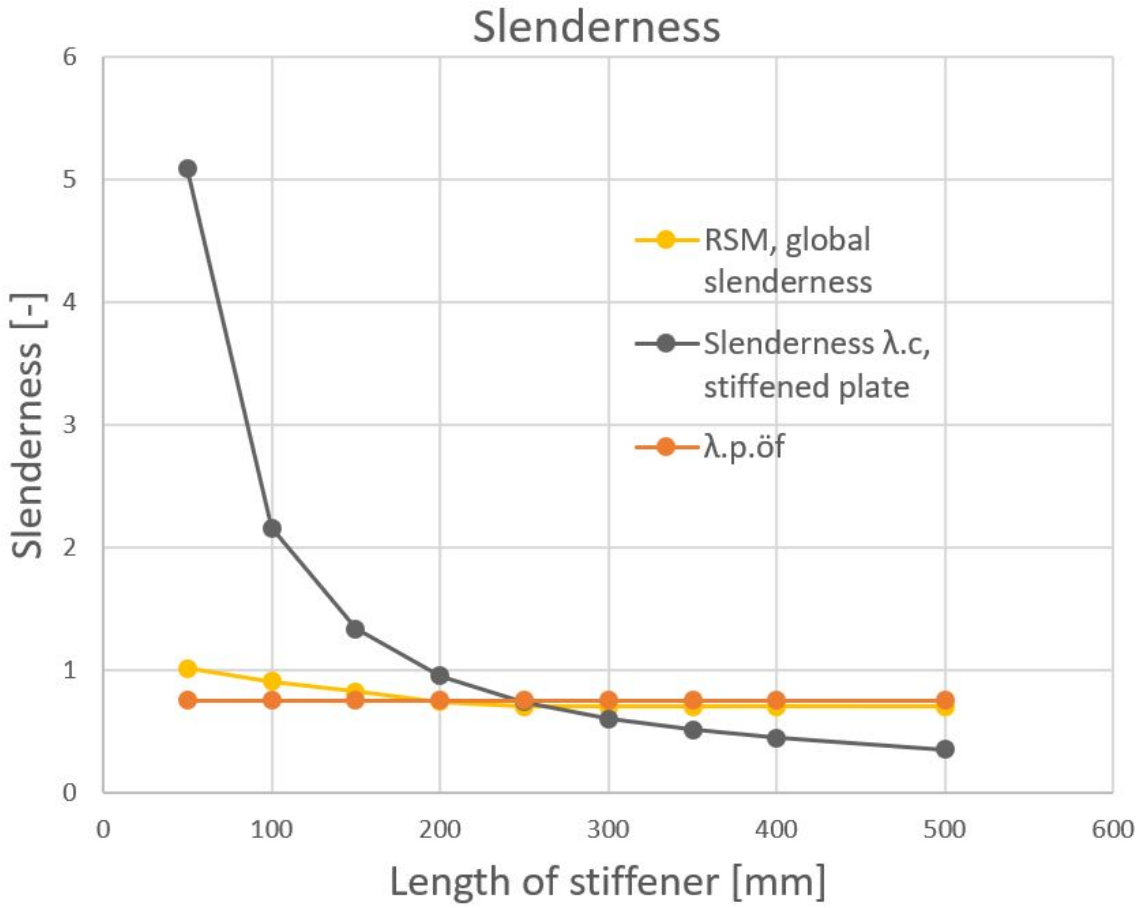


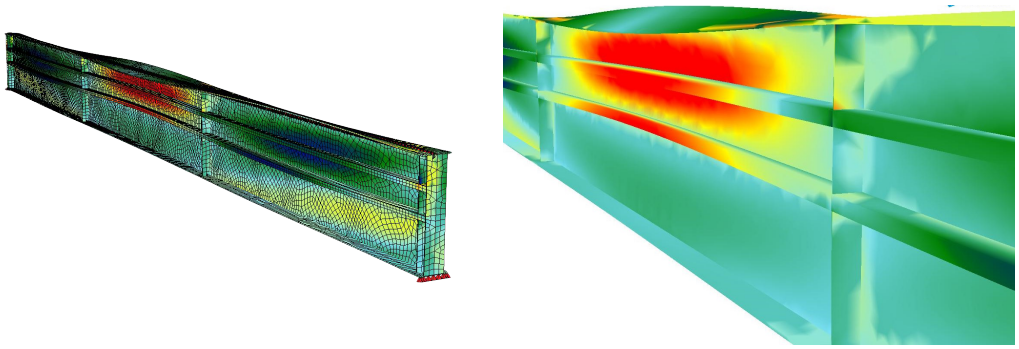
Figure 4.5: Slenderness

A lot of slenderness calculations are made in EWM but two of them are governing depending on the stiffener width. Its the slenderness for column buckling of the stiffeners and normal stress buckling of the top flange that are governing at different occasions. What is interesting about this is that the slenderness curve for RSM seems to change inclination at the point where the two EWM slenderness curves overlap. As if the global slenderness for RSM is influenced by the change in behaviour of the beam. Even though the inclination of the column buckling curve in EWM is a lot steeper than the part of the RSM curve in the same section.

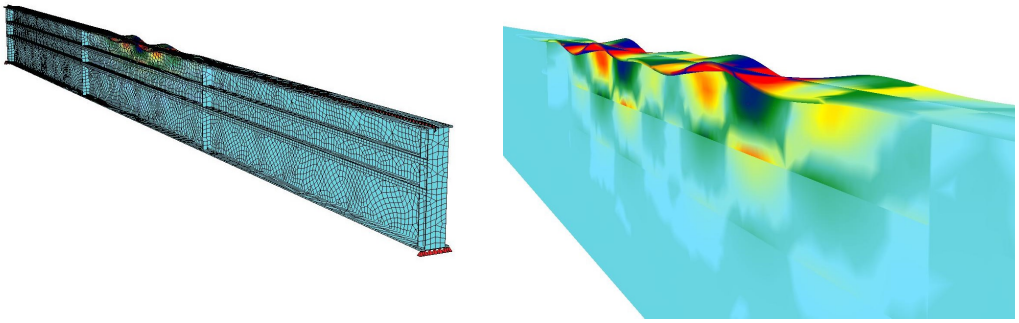
Even more interesting is that this point is close to the point where the inclination of the RSM curve in Figure 4.4 changes from one that follows the shear capacity curve to the moment capacity curve in EWM. So it seams that there is a relation between slenderness and buckling behaviour in both method as well as a relation between buckling behaviour and capacity growth in RSM. The FE buckling analysis is presented in the next section to see if the slenderness is describing the buckling behaviour correctly.

4.2.2 Buckling analysis

The width of the stiffeners are changed in order to achieve different buckling modes, which resulted in two governing buckling modes as presented in figure 4.6 below. The first buckling mode, which occurs for smaller stiffener widths, is however buckling of longitudinal stiffeners with the traversal stiffeners as supports. The next buckling mode is buckling of the top flange in compression and it occurs when the longitudinal stiffeners are wider. This is in agreement with the two governing stiffness parameters in EFW.



(a) Buckling mode a



(b) Buckling mode b

Figure 4.6: Two different buckling modes of the bridge with varying stiffener height.

Buckling mode	Buckling behaviour	Long. stiffener width [mm]
(a)	Column buckling of stiffeners.	50, 100, 150, 200, 250
(b)	Normal stress buckling of compression flange.	300, 350, 400, 500

Table 4.3: Buckling modes of the I-beam with varying stiffener heights.

The change in buckling behaviour seems to occur between the 250 and 300 mm stiffeners which coincide quite well with the stiffness curves in Figure 4.5. Note that the calculated slenderness values are in no way interpreted in the FE analysis which makes it even more interesting that they are so closely related. The visualized buckling behaviours in 4.6 makes it easier to understand why the capacity curve for RSM follows the EWM shear capacity curve for smaller whidts and the EWM moment capacity curve for wider stiffeners. Not only beacuse buckling mode a occurs in the web and buckling mode b in the top flange but mode a looks a lot like shear buckling as well. It might even be that the mode contains some shear buckling behaviour since combined buckling modes are possible in the analysis. This will be the case in the next analysed structure where a lot of sub plates and behaviours are close to each other.

5

Box section

This chapter covers the method comparison on the third and final structure. It is a section of an actual walking bridge which is under development at Ramboll as this report is being written. The bridge is called Oceanpiren and will be located in Helsingborg, Sweden. It is a curved box girder bridge which is stayed with cables connected to two steel pylons. Some simplifications will be made in the design and structural behaviour to be able to preform a parametric study within the time frame.

5.1 Method

The box girder bridge is studied with the the EWM analytically and compared with the RSM in combination with FEM. A stress- and buckling analysis with the FE-program SOFiSTiK is preformed just as with the I-beam in chapter 4. The analyses generates dimensionless eigenvalues and Von Mises stresses which are used as input data in the RSM hand calculations. The uniformly distributed load, Q_d, is then changed until the utilization is just below one. The bridge is then investigated with varying measurements of the sub parts of the bridge in order to achieve a deeper understanding of how the two methods are related to each other with different geometries. This parametric study is preformed with a parametric design tool in Grasshopper.

5.1.1 Description

The box girder bridge planned at Oceanpiren in Helsingborg is a walking and cycling bridge. A cable-stayed bridge with a s-shaped bridge deck as in figure 5.1

Figure 5.1: Oceanpiren

A cross section of the box girder bridge is presented in figure 5.2 below. There are vertical stiffeners both in the top plate and the inclined web plates to stiffen for both positive and negative bending moments. The top plate and the webs are extended past the side plates and connects with the end inclined end plates. Transverse stiffeners connects the top plate, side plates, webs and transverse stiffeners along the span. The holes in the transverse stiffener at the side plate corners and longitudinal stiffener ends are there to prevent stress concentrations and therefore increase the fatigue life. The top plate is inclined by 1.5% from the middle to the side ends for drainage purposes.

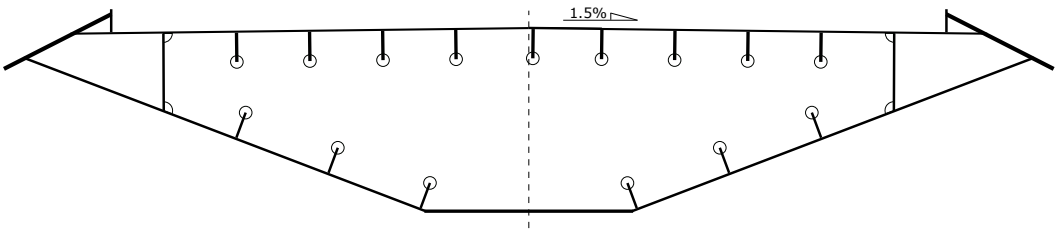


Figure 5.2: Cross section of box girder

The actual cross section dimensions varies along the span but a typical section is used as starting point for the calculations. The layout of the cross section is as previously mentioned simplified, the extensions of the top plate and webs are drawn back to the side plates. The holes in the transverse stiffener and the inclination of the top plate is ignored. These simplifications will not affect the structural behaviour with regard to short time buckling strength which is covered in this thesis.

The bridge span is reduced to a 18 meter, straight, simply supported box girder with dimension as presented in figure 5.3 and table 5.1 below. The inclination of

the webs are 21.12° from the bottom plate as the actual design. The transverse stiffener distance s_{ts} is constant at 2 meters through out the span except for by the supports where an extra stiffener is placed at a distance, s_{ts0} 0.5 meter from the support. The material properties which varies with the thickness are presented in Table 5.2.

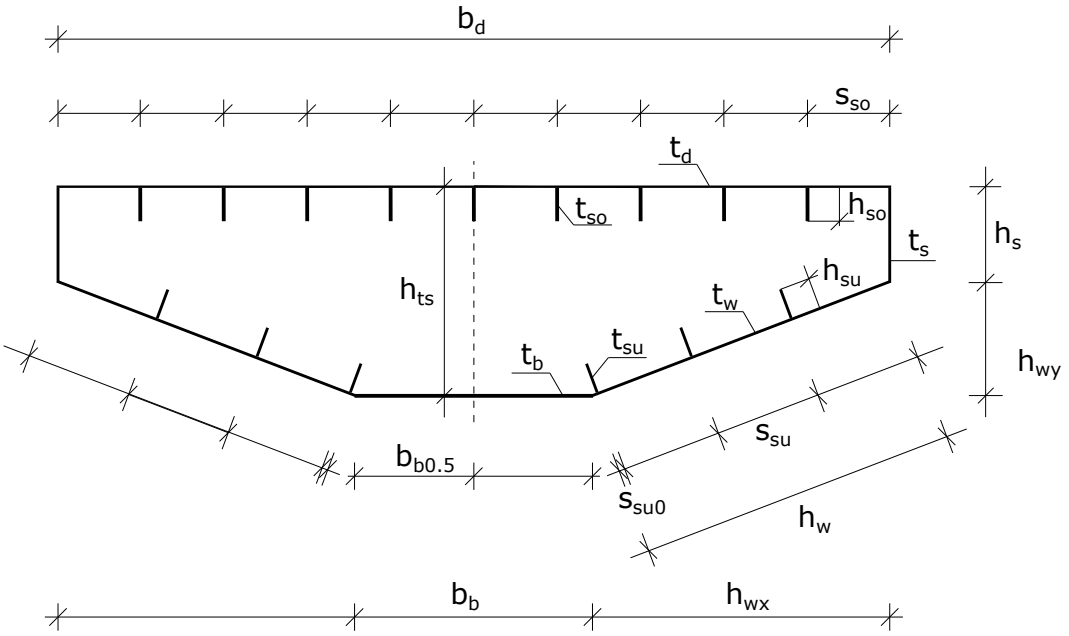


Figure 5.3: Simplification of cross section

Measurements of the beam			
	Length [mm]		Length [mm]
b_d	3500	s_{su0}	20
s_{so}	350	s_{su}	440
t_d	10	t_{su}	12
t_{so}	16	h_{su}	120
h_{so}	140	b_b	1000
h_s	389.2	t_b	16
t_s	12	t_{ts1}	18
t_w	12	t_{ts2}	12
h_w	1340	t_{ts3}	8
h_{wy}	482.8	s_{ts}	2000
h_{wx}	1250	s_{ts0}	500

Table 5.1: Measurements of the beam

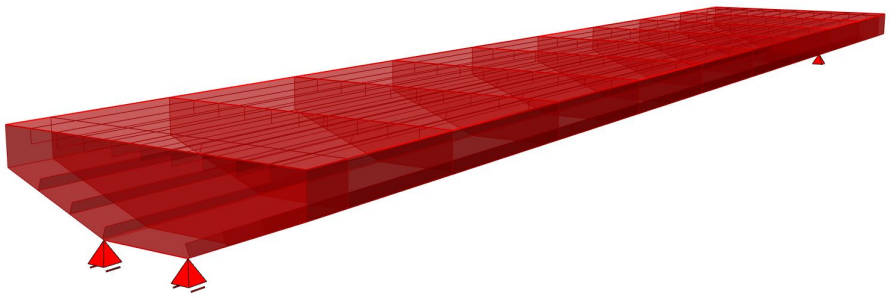
Plate with thickness:	**f_y [MPa]**	**E [GPa]**
t<40mm	390	210
t>40mm	335	210

Table 5.2: Material properties of the beam

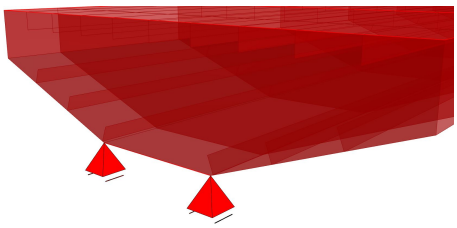
5.1.2 FE-Model

The geometry and boundary conditions are generated with Rhino with its algorithmic modeling plugin Grasshopper. The line along the supports are fixed in all directions on both sides except for one side which is free to move along the bridge in x-direction. The geometry is then interpreted in SOFiSTiK as a 3D system with a combination of quadrilateral and triangular shell elements for the analyses.

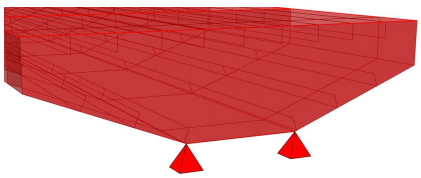
The structural model as well as boundary conditions are presented in 5.4 below. The supports are created with support lines placed at the very end of the sort edges of the bottom plate. No other boundary conditions are needed on this structure in order to obtain sufficient stability in the structure and prevent from instability problems, in contrary to the I-beam in Chapter 4. The layout of the transverse stiffeners along the span can be seen trough the transparent plate element in Figure 5.4a below.



(a) FE model of the box girder



(b) Left BC



(c) Right BC

Figure 5.4: FE-Model of the bridge girder

The stress analysis is a linear elastic analysis where the maximum Von Mises stresses are calculated for the most critical parts of the structure. An advantage of using a linear analysis is that a unit load can be applied to the structure and the final load capacity can be interpolated without rerunning the program. The buckling analysis is a material-linear (but geometrical-non-linear) eigenvalue calculation which is solved with a simultaneous vector iteration. The linear nature of this procedure

makes it possible, just as the stress analysis, to use a unit load which is then interpolated up to the ultimate capacity.

5.1.3 Convergence study

The convergence study on the box section is preformed by studying both the Von mises stress in a single point in the middle of the girder as well as the buckling eigenvalue as the mesh size decreases. The same load is applied for every mesh refinement and the outcome is plotted and presented in Figure 5.5 and 5.6 below.

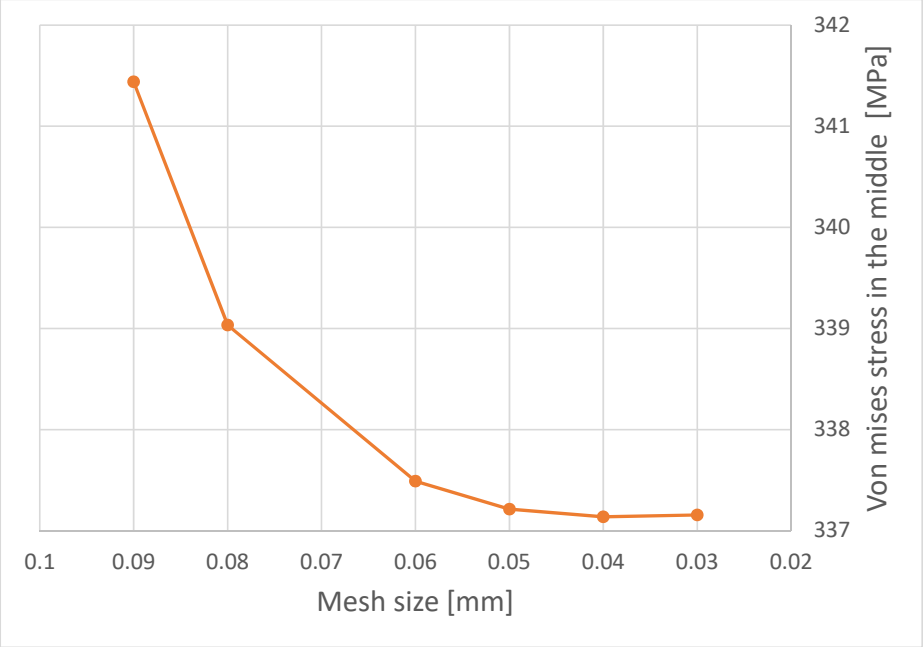


Figure 5.5: Von mises convergency of the boxsection

An average element size of 60 mm seems to be sufficient for the Von mises stresses by just analysing the slope of the curve since no significant change in the result occurs below that. The percentage difference is although below 1 % already between 90 and 80 mm which indicates that the stress analysis is not that sensitive to the mesh size. It is indeed important in this model to not refine the mesh more than necessary because of the already vast computational time so a mesh size of 90 mm is sufficient for the stress analysis. The analyses are computed on the same model so the other convergence study could be governing.

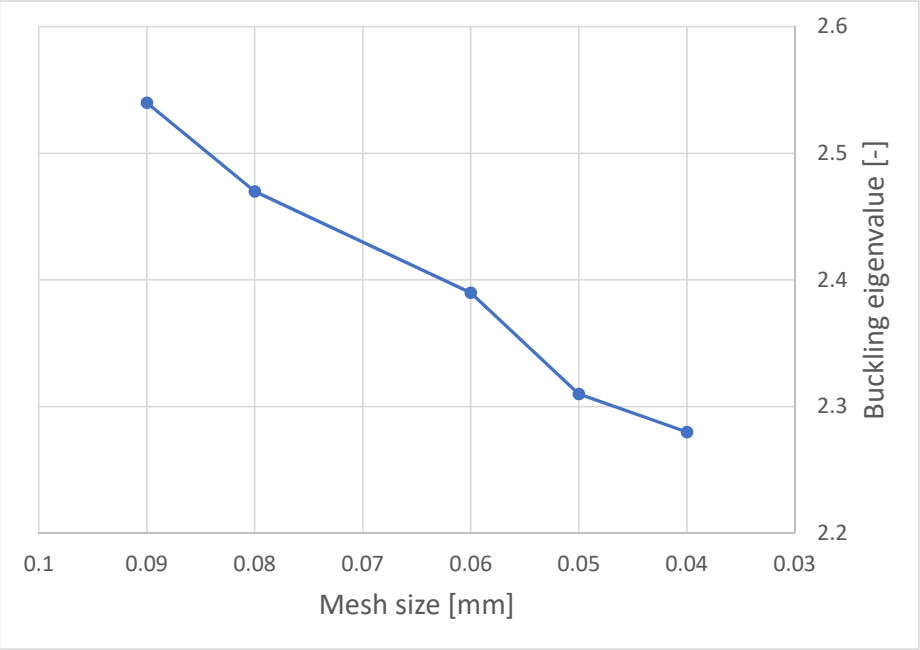


Figure 5.6: Egenvalue convergency of the boxsection

The buckling eigenvalue does not show much of a convergence pattern before the 50 mm element size. The calculation time reached about 40 minuets for the 40 mm mesh size which should be regarded as a maximum limit since the analysis should run about 50 times so the refinement stopped there. The slope does decrease a bit between the element sizes 50 and 40 mm but the percentage change is 1.3%. The 1% convergence limit is not reached but the computational time limit is, hence a mesh size of 40 mm has been chosen for the model.

5.1.4 Parametric study

Rhino and the plug-in tool grasshopper is used to effectively generate a geometry where parameters easily can be changed. The geometry is then analyzed with a stress analysis and a buckling eigenvalue analysis in the software Sofistik. Both analyzes are linear so that a unit less load can be applied. Since the geometry most often changes during projects there is a need of a general code with the possibility to make easy changes regarding for example its measurements and amount of stiffeners. A parametric tool which can be used for different box girder bridges can streamline a designers workflow significantly and more room for optimization is possible.

The design tool made in grasshopper is not fully generalized but merely quite adapted to the parametric study preformed in this project. Other design changes are possible but the following parameters will be investigated in this thesis:

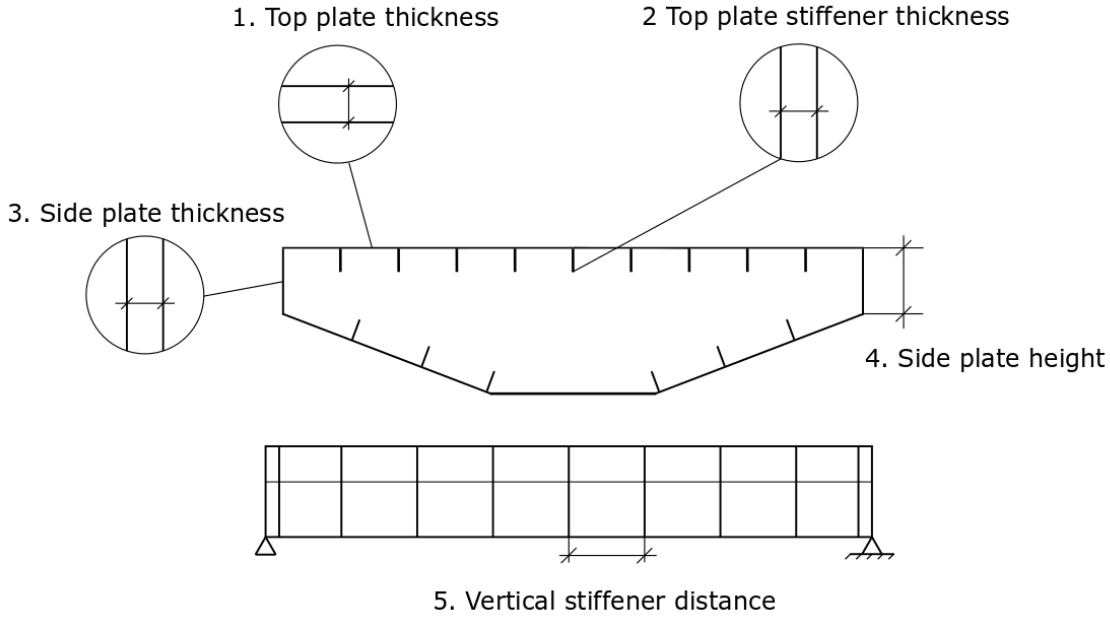


Figure 5.7: Illustration of the studied parameters

Figure 5.7 illustrates which parameters that will be investigated and in what order. Since the structure is simply supported and subjected to a uniformly distributed load only the plates above the center of gravity will be in compression. The study is concentrated on those plates since they will affect the capacity directly which is not the case for example the bottom plate.

The measurements that should be investigated for every parameter are presented in table 5.3 below. The parameters are investigated one by one while the others are set to the original measurements. The original measurements are presented in bold in the table. The top plate is in cross section class 4 in the original design but as the thickness increases the class changes to 3 for the whole section. That means that

the calculation methods are unnecessary since no reduction will be needed. That investigation is only meant to verify that the two methods approaches each other in that section. The top plate is in cross section class 4 for all other studies so the whole cross section is to, regardless of the other measurements.

1. Thickness of the top plate [mm]								
4	6	8	**10**	12	14	16	18	

2. Thickness of the top plate stiffeners [mm]								
2	4	6	8	10	12	14	**16**	18

3. Thickness of the side plates [mm]								
1	3	4	6	8	10	**12**	14	16

4. Height of side plates [mm]								
389.2	470	550	630	710	790	870	950	1030

5. Amount of vertical stiffeners							
5	6	7	**8**	9	10	11	12

Table 5.3: Investigated parameters and thicknesses

5.2 Result & Discussion

The already mentioned parameters with different measurements in table 5.3 are investigated with the EWM and the RSM. Generally the methods coincides quite well, but for some thicknesses there are a significant difference in capacity and a changed behaviour in the buckling analysis. The results from the different parameters are presented one by one in section 5.2.1 to 5.2.5 below. Both the capacity and the buckling analyses are discussed thoroughly for every parameter.

5.2.1 Top plate thickness

The thickness of the top plate has quite a large impact on the capacity of the bridge and the buckling behaviour. The capacities of the box girder are compared with varying top plate thickness calculated with the EWM versus the RSM with eigenvalues and von mises stresses from Sofistik.

5.2.1.1 Capacity comparison

The original box section consist of a 10 mm thick top plate which resulted in a lower capacity with the RSM compared to the EWM according to figure 5.8. Both moment- and shear capacity are plotted for EWM as with the I-beam in chapter 4 but the interaction is not relevant for in this structure so that is left out. The plot includes vertical lines which marks the boundaries of the different buckling modes,

a, b and c. The buckling modes and their influence will be presented in the next section.

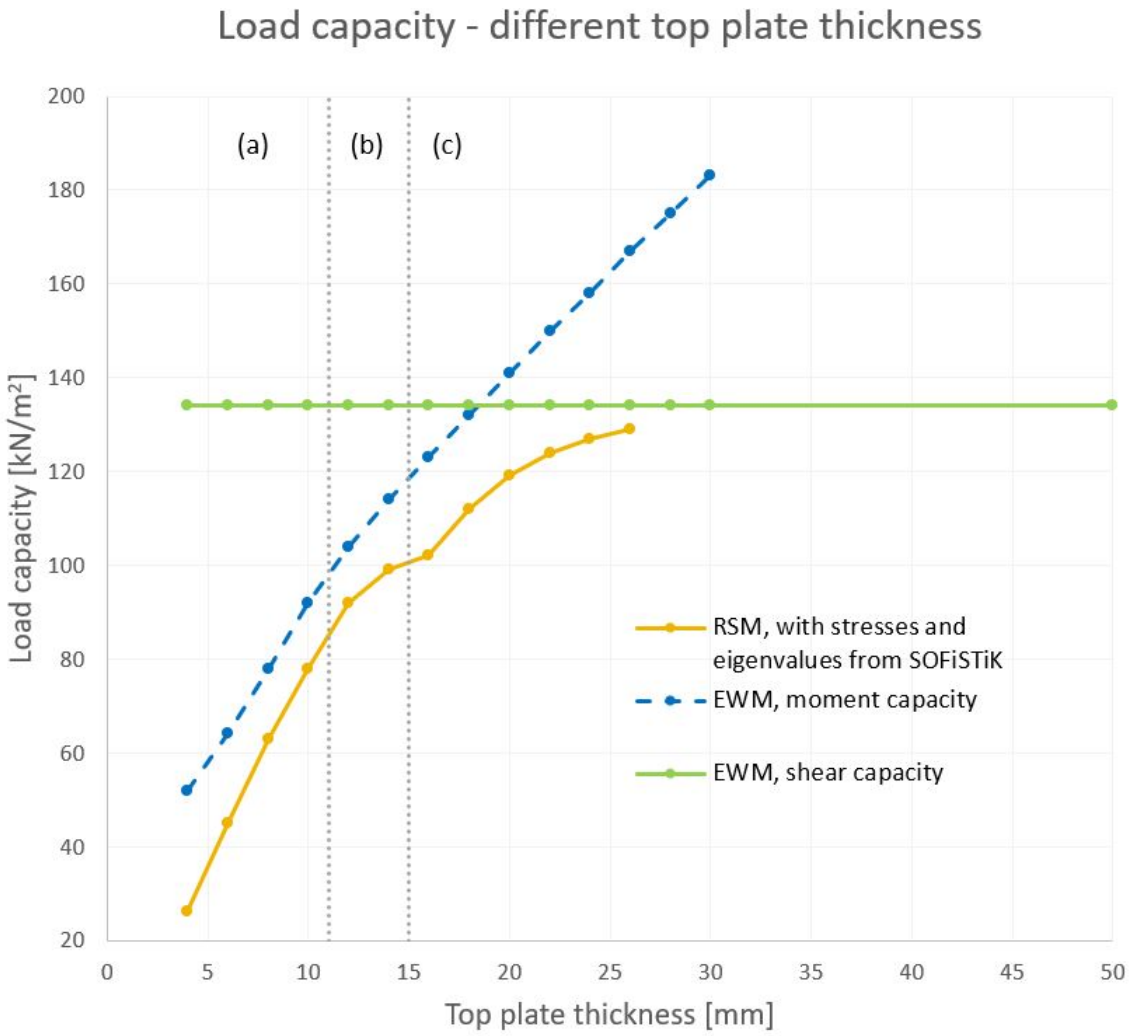


Figure 5.8: Load capacity of the box girder bridge with the EWM and the RSM with varying top plate thickness.

The yellow curve which represents the capacity with the RSM is, as mentioned before, lower bound for all thicknesses of the top plate. The curve does follow the blue moment capacity curve from the EWM for thinner top plates and the green shear capacity curve for thicker top plates. This seems reasonable since the two EWM curves cross each other in the middle. But the yellow RSM curve is furthest away from the two EWM curves close to that intersection as if the method is less accurate when the governing force changes. This less accurate part also happens to be within the limits of buckling mode b so the answer might be in the behaviour of the structure. The I-beam in chapter 4 had a close relation between its buckling mode and governing slenderness curve. The slenderness curves have been compared for this structure as well and the result is presented in figure 5.9 below. Just as with the capacity plot, two vertical lines are added to show where the buckling mode changes.

Two slenderness curves are very close. One of them is corresponding to plate like buckling of an equivalent plate, which basically means a fictional plate which is equvalent to the combination of the top plate and all longitudinal stiffeners conected to it. The other slenderness curve is corresponding to column buckling of the longitudinal stiffeners. It does not matter mathematically which one is used in the calculations since they are almost the same but the buckling analysis will reveal which one that corresponds to the actual behaviour.

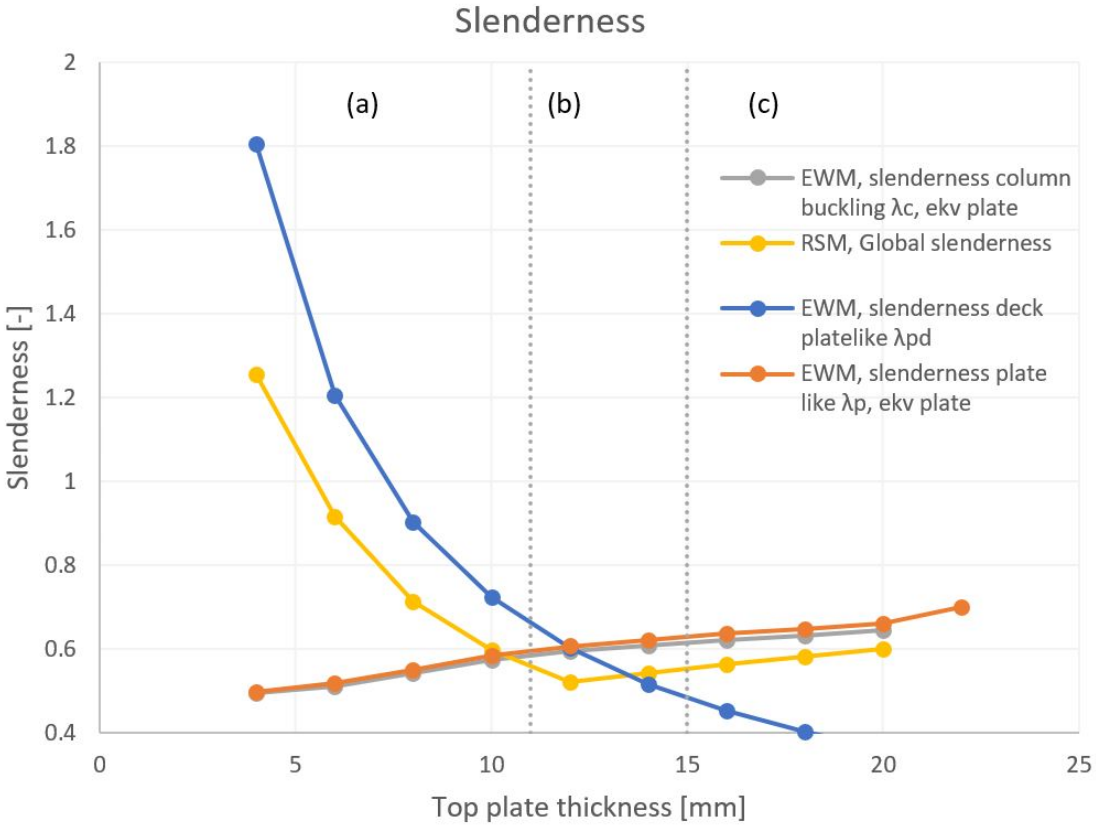


Figure 5.9: Slenderness with varying top plate thickness. Buckling mode a, b and c.

Besides the two very close slenderness curves in the EWM there are a third which is governing for the thinner top plates. This is a slenderness that corresponds to a situation where a part of the top plate buckles between two stiffeners. This is a quite special situation which occurs when the top plate is so thin compared to the stiffeners that they wont help stiffen it up. The stiffeners acts in stead as supports where the top plate is free to move in between.

The non-governing parts of the EWM slenderness curves has been removed in figure 5.10 below to make it easier to compare them to the RSM slenderness curve. The grey curve that represents column buckling of the stiffeners is entirely removed due to its similarity to the slenderness curve of the equivalent plate.

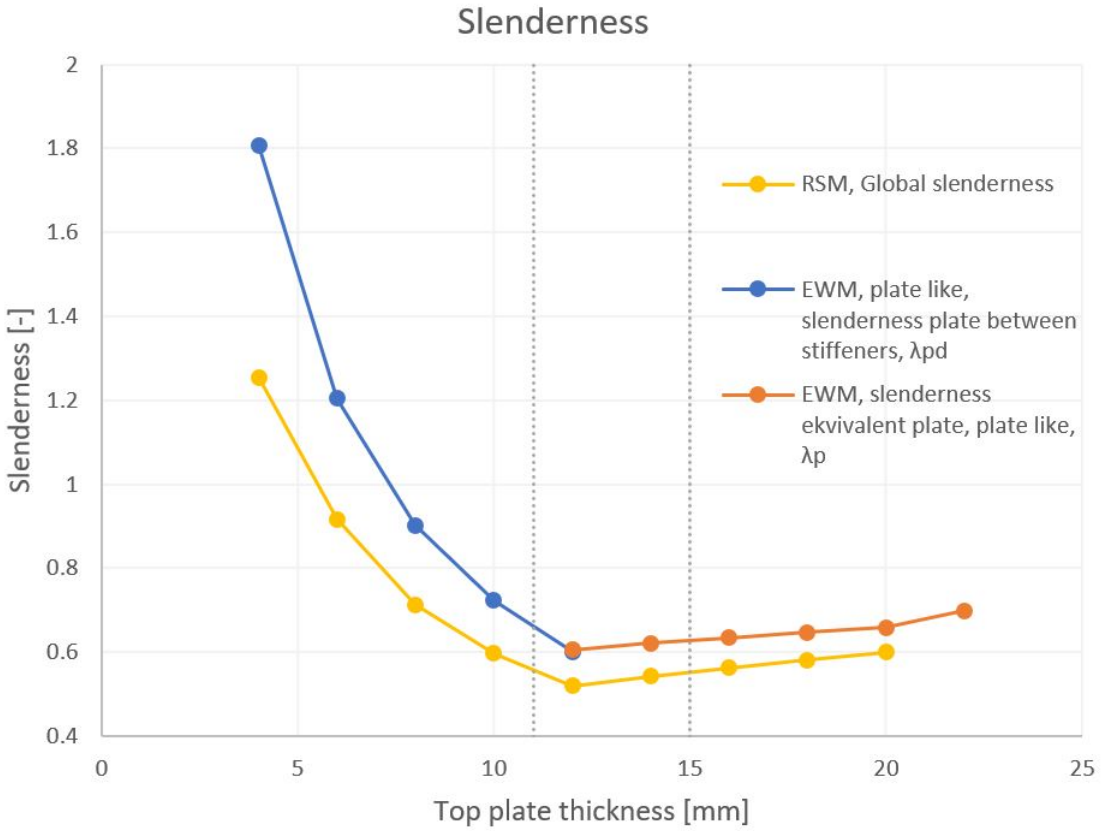
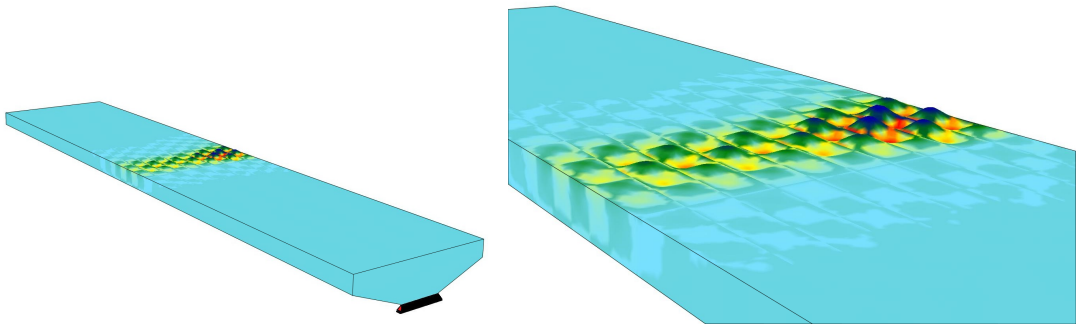


Figure 5.10: Slenderness with varying top plate thickness. Buckling mode a, b, c

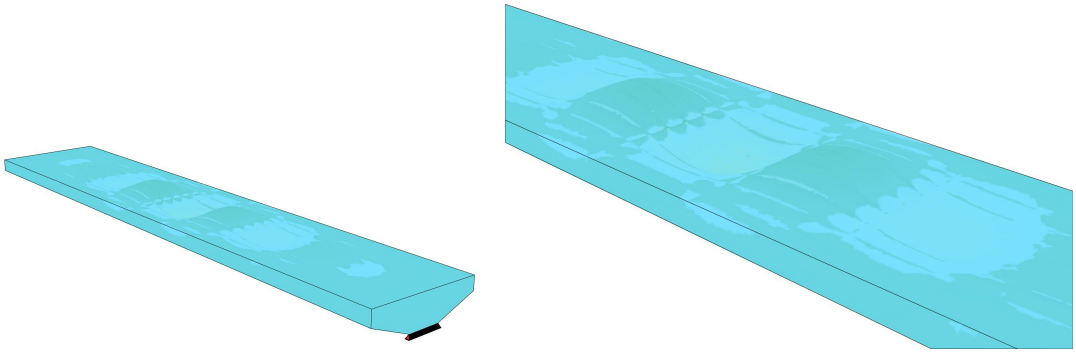
The combined EWM curve and the RSM curve are now looking very similar to each other. The RSM curve is, just as the capacity, below EWM for every thickness but a little bit closer for thicker top plates. The increased difference in capacity that occurred in the middle of the span earlier observed in figure 5.8 does not seem to have much to do with the slenderness. No such patterns occurs in the slenderness comparison and the slenderness curves does not change between buckling mode b and c. It might therefor be that the slenderness and the buckling modes are'nt that close related for this structure and that the actual buckling mode can tell more about why there is a gap in the middle of the capacity plot.

5.2.1.2 Buckling analysis

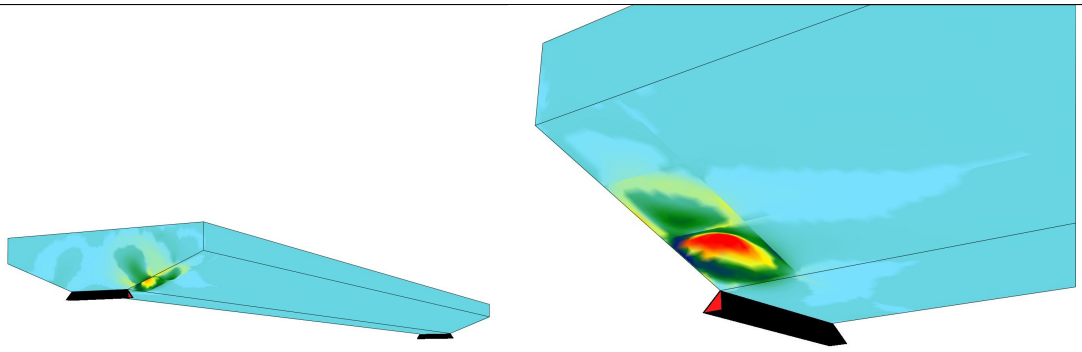
The structure buckles in three different ways through out the parametric study of the top plate thickness. The buckling modes are presented in figure 5.11 below. For the thinnest top plates (4-10 mm) the top plate buckles locally between stiffeners by plate like buckling (figure 5.11a). The second buckling mode (figure 5.11b) at the thicknesses 12 and 14 mm is global plate like buckling of the top plate and the longitudinal stiffeners together. The third and final buckling mode (figure 5.11c) occurs at top plate thicknesses 16 mm and over. It is shear buckling of the web between the two end stiffeners by the support.



(a) Buckling mode a



(b) Buckling mode b



(c) Buckling mode c

Figure 5.11: Three different buckling modes with varying top plate thickness.

To get a clear view of the results, the buckling behaviour for each top plate thickness with its respective buckling mode are presented in table 5.4 below.

Buckling mode	Buckling behaviour	Top plate thickness
(a)	Local normal stress buckling of the top plate.	4, 6, 8, 10 [mm]
(b)	Global normal stress buckling of the top plate and stiffeners.	12, 14 [mm]
(c)	Shear buckling of the web.	16, 18 [mm]

Table 5.4: Buckling modes with varying top plate thickness.

Buckling mode a and b corresponds well to the governing slenderness curves obtained by the EWM but mode c does not. No slenderness that corresponds to shear buckling is governing in the EWM calculations. It is though within buckling mode c that the shear capacity is starting to be governing so it would be logical for the shear buckling slenderness to be governing in that area. But shear capacity, shear buckling and shear buckling slenderness are three different things that not always relates entirely. The buckling analysis seems, in this case, to have anticipated a buckling mode where the hand calculated did not. The calculations are based on stiff end supports which the design assures according to the codes but the buckling mode should maybe not occur between the end stiffeners in that case. The two methods agrees quite well within this buckling mode after all so even though this is odd, the capacity miss match can not depend on this. A guess could instead be that the methods differ the most when the behaviour of the structure changes vastly. In this case, from buckling in the top plate in the middle of the span to shear buckling in the web in the end of the span. The moment- and shear capacity are calculated separate in the EWM but in the RSM a combined capacity, based on the whole structure, is calculated. This could be an affecting factor to why the method is more conservative in the section where the change of behaviour occurs.

5.2.2 Top plate stiffener thickness

The original thickness of top plate stiffeners is 16 millimetres. In this section thicknesses between 5-18 millimetres are studied. The top plate stiffener thickness has a large impact on the behaviour of the box girder bridge because of it's stiffness contribution to the top plate which is the weakest part in the original design.

5.2.2.1 Capacity comparison

The stiffness contribution from the stiffeners to the already weak top plate decreases as they decrease in size which is presented in figure 5.12 below. The shear capacity is never governing in this study because its not effected by the decrease of stiffener thickness. As for the previous investigation the RSM is lower bond for every calculation in the study. It occurs three different buckling modes which are further discussed in the next section but the boundaries are presented in figure 5.12 and labeled a, b and c.

The capacity difference between the methods differs from 24 to 14% from a 6 to a 18 mm stiffener thickness which indicates that the methods gets closer when the stiffeners increase in thickness. There are two reasons to this, firstly the cross section gets closer to a class 3 section so that the reduction with regard to buckling approaches zero in both methods. This was the same case in the previous section where the section actually reached class 3 for the thickest top plate measurements. The methods did in fact not receive the exact same capacity within the class 3 cross section. The other reason is that the top plate, with its connecting stiffeners, is the weakest part of the cross section in the original design. So an increase in both top plate thickness and stiffener thickness should result in a equalization in capacity

between the sub plates within the section. As mentioned in section 2.2.1 the EWM lack the ability to account for load shedding between sub plates within the section so a more even capacity distribution would decrease the impact of this inability.

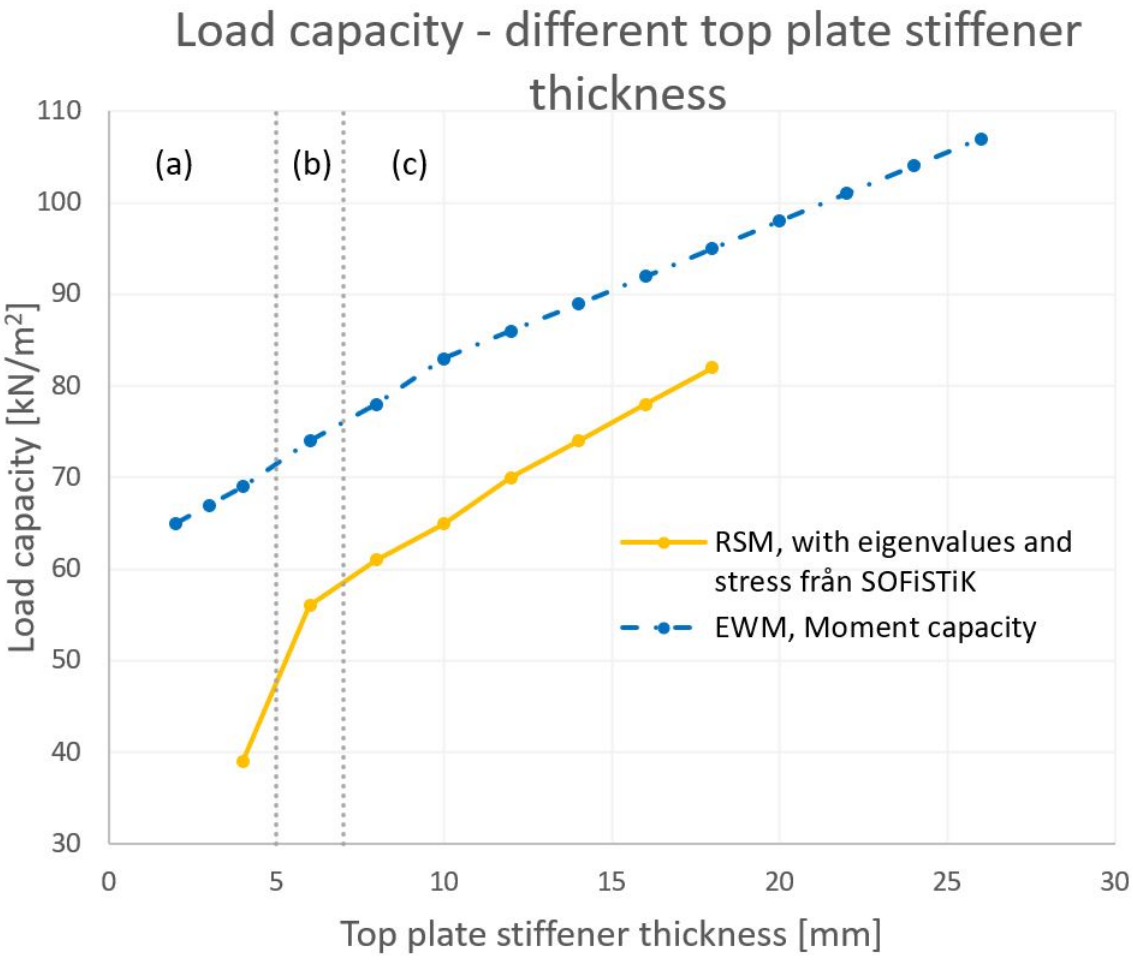


Figure 5.12: Load capacity with EWM and RSM with eigenvalues from Teddy for different top plate stiffener thickness.

The capacity drops significantly in the RSM calculation compared to the EWM when the top plate stiffener is set to a 4 mm thickness. This might be related to the buckling mode since this thickness is the only one within buckling mode a. The slenderness have been plotted in figure 5.13 for both methods to compare and see if this might explain this capacity drop. The EWM has two governing slenderness curves for this investigation. The blue curve which is governing from 4-12 mm stiffener thickness is related to local buckling of one stiffener. The orange curve which is governing for 14 mm stiffeners and thicker is related to buckling of the equivalent plate. The later of the two correlates well with the RSM slenderness curve except for a flatter inclination between 4 and 6 mm stiffeners. The blue curve which is a lot steeper is governing through all three buckling mode changes so the capacity drop at 4 mm stiffener thickness does not seem to have much to do with the slenderness. The difference in the slenderness between the two methods does in fact

not seem to correlate with the over all difference in capacity. A high slenderness should result in a weak section with high reduction due to the increased risk of buckling. So the very high slenderness in the EWM for lower thicknesses does not affect its capacity very much since the capacity in the EWM is quite steady through out the whole study. Since the slenderness doesn't correlate very well to the buckling modes the answer could lay within the buckling analysis in the next section.

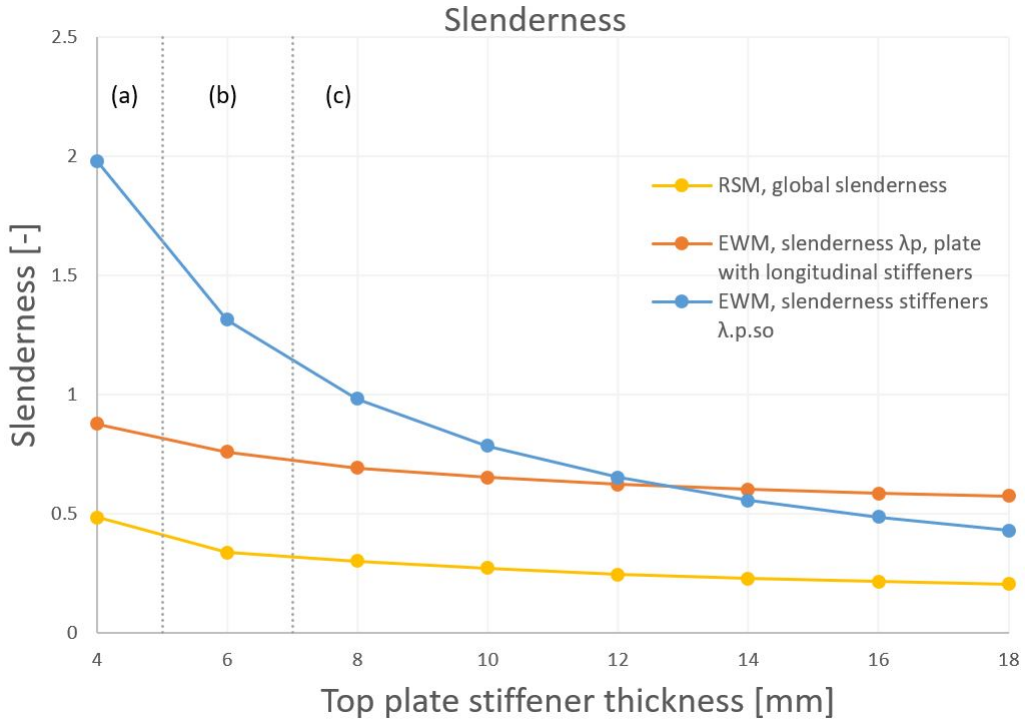
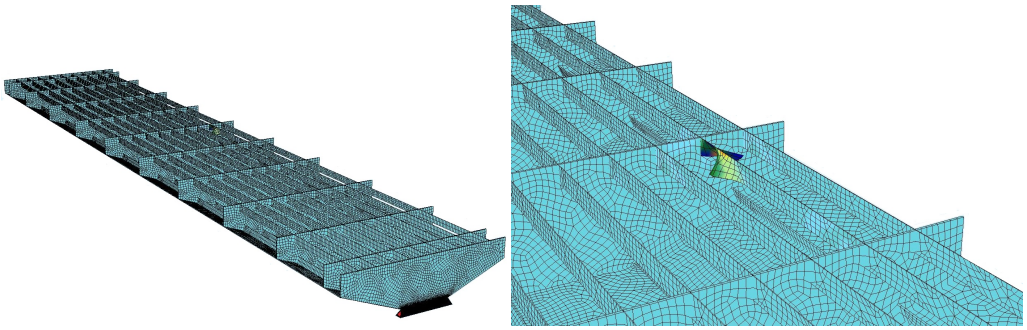


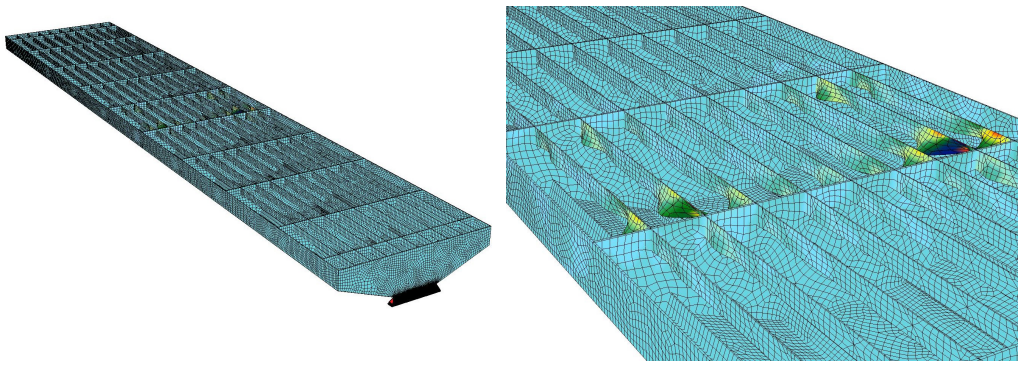
Figure 5.13: Slenderness with different top plate stiffener thicknesses.

5.2.2.2 Buckling analysis

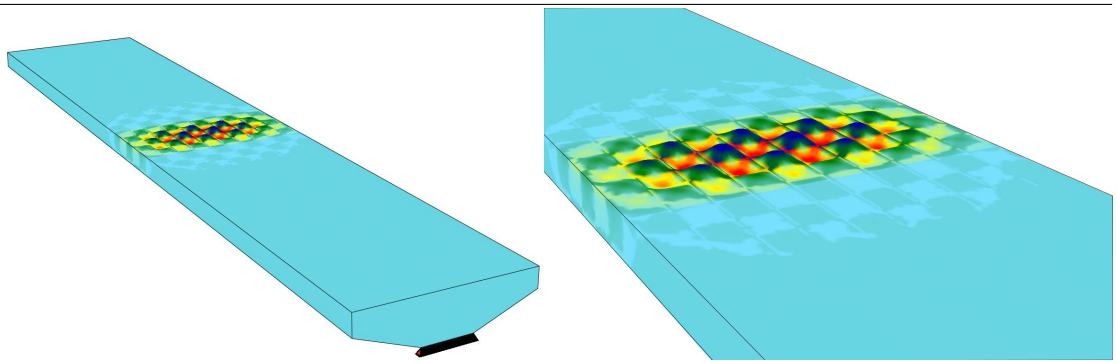
The buckling analysis resulted in three buckling modes as mentioned before, these are presented in figure 5.14 and table 5.5 below. All three modes occur in the center of the span. The first buckling mode, mode a, is local buckling of one longitudinal top plate stiffener. The second mode, buckling mode b, is local buckling of several top plate stiffeners and the third mode, mode c, is normal stress buckling of the top plate. This correlates to the two slenderness curves obtained in the previous section but the slenderness related to buckling of one or several stiffeners is governing through out all three of the buckling mode so the overlaps does not match each other very well. The mesh is kept in figure 5.14a and 5.14b to be able to see the contours of the top plate stiffeners which otherwise blends in with the surrounding plates.



(a) Buckling mode a



(b) Buckling mode b



(c) Buckling mode c

Figure 5.14: Three different buckling modes of the bridge with varying top plate stiffener thickness.

Buckling mode	Buckling behaviour	Thickness [mm]
(a)	Normal stress buckling of one longitudinal stiffener.	4
(b)	Normal stress buckling of all longitudinal stiffeners.	6
(c)	Normal stress buckling of the top plate.	8, 12, 14, 16

Table 5.5: Buckling modes with varying top plate stiffener thickness.

One could argue that buckling mode a and b are the same buckling mode but since something changes in the capacity between these two modes it may be wise to separate them. Both modes only consist of one thickness each and the rest of the thicknesses belongs to mode c so this only occurs when the stiffeners are very thin in comparison to the rest of the cross section. A possible reason for the big capacity difference when the stiffeners are as thin as possible actually has to do with how it buckles. In the EWM a check for the stability of the stiffener it self needs to be fulfilled. The stiffener needs to be prone to torsional buckling which is not the case for the 4 mm stiffener since the check is not passed and it's clear in the buckling analysis that the stiffener experiences torsional buckling. The check is actually not passed for the 6 mm stiffener either but since the buckling mode is spread out through all stiffeners, it seems that the instability problem does not really come in to play so to speak. The EWM does however not account for this instability of the stiffener so the capacity should maybe be decreased but this is not suggested in [SIS, 2011b] since a stiffener that does not pass the check is quite useless and should be avoided. In the RSM however there is no such suggested design check to be made so the weak stiffener is accounted for which probably is the reason for the earlier mentioned difference change between the two methods capacities.

5.2.3 Side plate thickness

The original side plate thickness is 12 millimetres. The capacity is compared for thicknesses between 1-16 millimetres. The side plates has great impact of the shear capacity so this investigation will probably result in a change of governing capacity in the EWM calculation.

5.2.3.1 Capacity comparison

The results from the side plate thickness study is different compared to the previous results since the RSM is higher bond for some thicknesses which has not been the case before. The result is presented in figure 5.15 below. Vertical lines have been included to show the buckling mode changes throug out the study, just as for the previous results. Once again three different buckling modes has been identified, thieve been labeled a, b and c and will be further discussed in the next section.

The RSM gives higher capacity for almost all thicknesses where the shear capacity is governing in the EWM, which is at side plate thicknesses from 1 to 8 mm. The moment capacity is governing for side plates thicker than that and the EWM gives higher capacity in that section. The shape of the RSM curve follows both EWM curves quite well which has been the case for the previous investigations as well. In section 5.2.1, where the top plate thickness where investigated, a change in governing capacity where also the case. But an inconsistency between the two method where identified in the section where the overlap between shear- and moment capacity occurred. This inconsistency does not occur in this investigation. The slenderness

curves or buckling analysis in this investigation may explain why that is not the case.

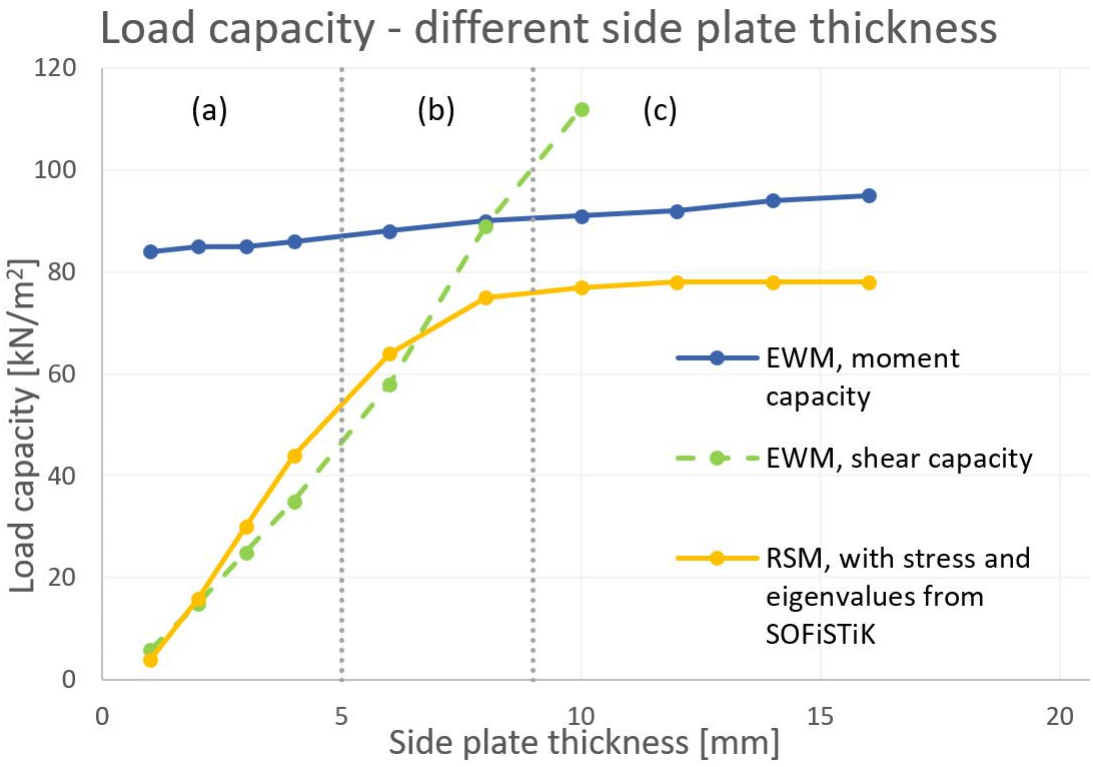


Figure 5.15: Load capacity of the box girder bridge with the EWM and the RSM with varying side plate thickness.

The slenderness are plotted for both methods and presented in figure 5.16 below. The EWM has two governing slenderness curves, slenderness for plate like buckling of the side plate for thinner side plates and slenderness for plate like buckling of the equivalent plate (top plate and longitudinal stiffeners) for thicker side plates. The side plate slenderness is almost governing for all thicknesses but slenderness of the equivalent plate is just a little bit higher from 10 mm side plates and higher. The yellow RSM slenderness curve is following the shape of the two EWM curves as it should but it's usually lower than EWM which is not the case in this investigation.

An extremely high slenderness factor of about 15 is measured at the thinnest side plate (1mm). The slenderness factor usually lays below 2 but both methods receives higher values than that for thicknesses below 3 mm. That is perhaps not very surprising since 3mm is very thin for a plate that is 2 m long. The capacity for those measurements are quite low as well so there is no doubt that such thin side plates are quite ineffective. There is not much of a change in the slenderness when the side plates are set to thicknesses at 8 mm and higher. This is also visualized in the capacity plot where no significant change to the capacity occurs when the side plates are thicker than 8 mm in both methods. This is interesting since they are 12 mm thick in the original design. It is although important to ad that this thesis only

focuses on short time buckling capacity and no other lode case than a uniformly distributed load is calculated so there is probably other affects to the side plates that governs the design.

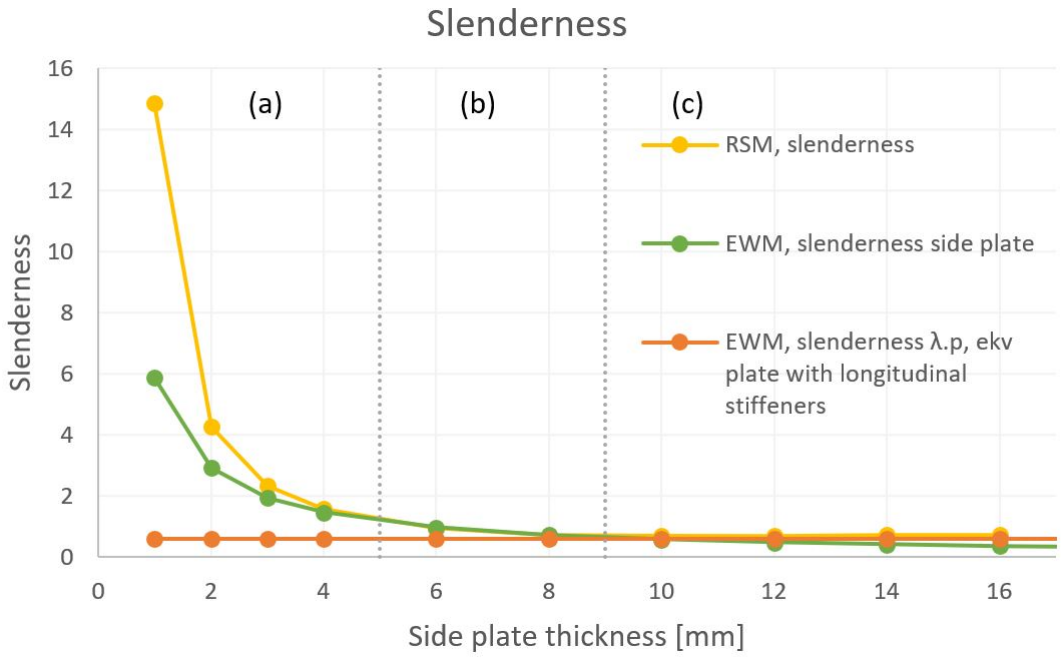
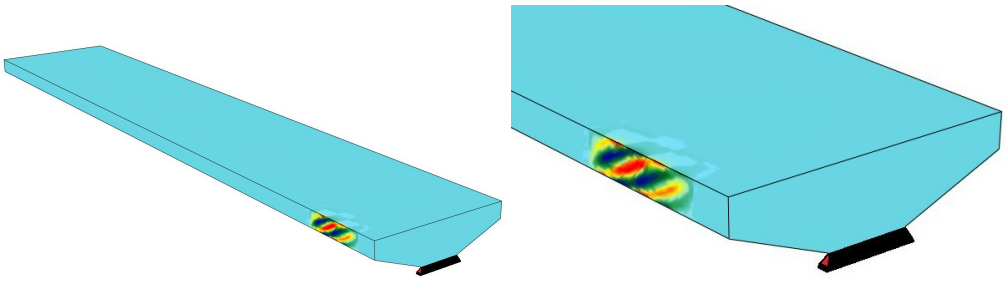


Figure 5.16: Slenderness with varying side plate thickness.

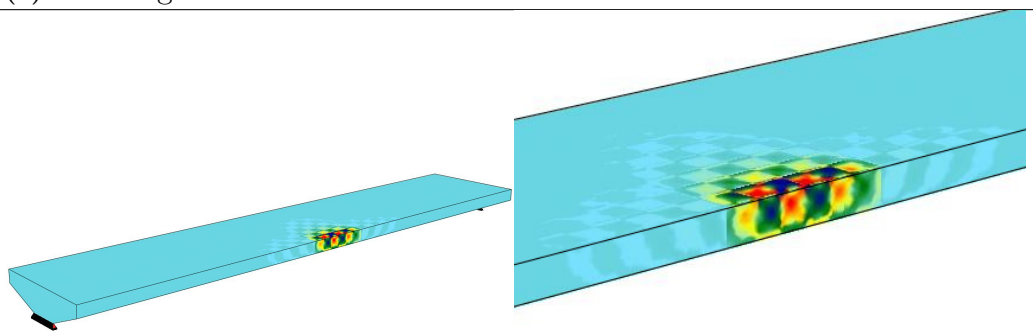
There is no inconsistency to the slenderness curves except for the very high slenderness in RSM for very thin side plates. But there seems to be nothing that explains the difference from this investigation to the top plate investigation where an increase in the capacity difference between the methods occurred in the overlap from moment- to shear capacity. perhaps the buckling analysis can reveal why this case is different to the top plate study.

5.2.3.2 Buckling analysis

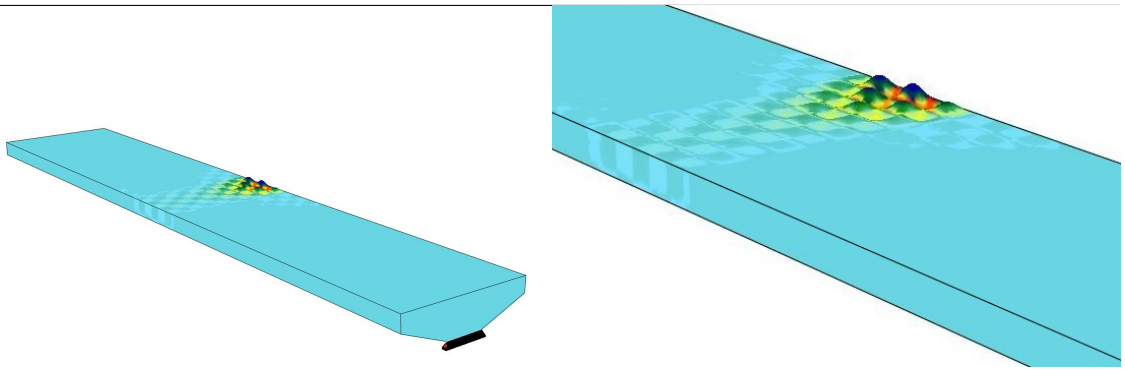
An interesting buckling behaviour is achieved with an varying side plate thickness as presented in figure 5.17 and table 5.6. Shear buckling near the support occurs with side plates thicknesses below 4 mm (mode a). Normal stress buckling of the side plate in the middle of the span occurs at thicknesses within 6-8 mm (mode b). The third and final buckling mode (c) occurs at thicknesses higher than 8 mm and is local buckling of the top plate between stiffeners in the middle of the span but towards one of the side plates. The third buckling mode moves towards the middle of the section as the side plate gets thicker but it is still normal buckling of the top plate.



(a) Buckling mode a



(b) Buckling mode b



(c) Buckling mode c

Figure 5.17: Three different buckling modes of the bridge with varying side plate thickness.

Buckling mode	Buckling behaviour	Thickness [mm]
(a)	Shear buckling of the side plate. of the side plate	1, 2, 3, 4
(b)	Local normal stress buckling of the side plate.	6, 8
(c)	Local normal stress buckling of the top plate.	10, 12, 16

Table 5.6: Buckling modes with varying side plate thickness.

There are some differences to the top plate study to be observed. Firstly the shear buckling mode occurs in the side plate in stead of the web. The side plate is totally

in compression and the web is mostly in the tension zone so the transition from shear buckling to normal stress buckling is quite abrupt. The web is also further away from the top plate where the next buckling mode occurs than the side plate. The buckling mode occurs after the second vertical stiffener from the support in stead of in between the two end stiffeners. This out rules the possible scenario where the assumption of a rigid end support in the EWM does not account for that buckling mode. Another difference is that the transition between buckling modes seems a little bit more smooth. In this case the change from shear buckling near the support to normal force buckling in the middle happens in the same sub plate. In the top plate study the change of buckling mode type was combined with a change of buckled sub plate. All buckling modes are local in this study as well which indicates some type of consistency.

All of the observed differences above can have more or less impact to why the side plate study gets a more consistent RSM capacity curve than the top plate study. These indications is of course hard to observe without a parametric study and a typical designer might not change the design to this extent. A designer is probably not able to see these transitions between buckling modes and weather they are smooth or not. It is therefore hard to conclude any guidance for where to use any of the methods from this particular study. But the methods are quite close in both the top plate study and the side plate study when shear force is governing in the EWM which could be a more helpful finding for this purpose. Perhaps the next two studies can be helpful to further investigate this.

5.2.4 Side plate height

The side plate height study is unique in the context since the change of side plate height results in a change of the total height of the cross section. The height ratio between the web and the side plate changes and the neutral axis moves from the web to the side plate. So the side plate which until now has been entirely in the compression zone will experience some tension as well. The original side plate height is 389.2 millimetres and this study covers heights up to 1030 millimetres which is almost three times as high.

5.2.4.1 Capacity comparison

The capacity curves are plotted and presented in figure 5.18 and as for most of the previous results, the RSM is lower bond for all calculated measurements. the boundaries of the buckling modes are once again added to the plot but this time there are four different buckling modes, a, b, c and d along the studied designs. The moment capacity in the EWM is governing for lower side plate heights but increases in a linear manner as the side plate increases because of the increase of moment of inertia within the cross section. The shear capacity in the EWM however behaves quite irregular where an increase in the capacity occurs for the lower side plates. The capacity starts to decreases as buckling mode b emerges and then flattens out along mode c and then increases a little bit again by mode d. So it seems that the

shear capacity in the EWM is quite close related to the governing buckling mode in this study, this will be further analyzed in the next section. The RSM curve, on the other hand, do not seem to be as influenced by the buckling mode. The inclination of the curve does changes by buckling mode b where the capacity flattens out a bit so there is some relation to the buckling mode there as well. The methods are closest when the side plate is as short as possible and furthest away when the EWM changes from moment- to shear capacity. Maybe the methods approaches each other again for side plates higher than 1030 mm but the capacity increase wearers of in both methods so the design gets less material effective for higher side plates.

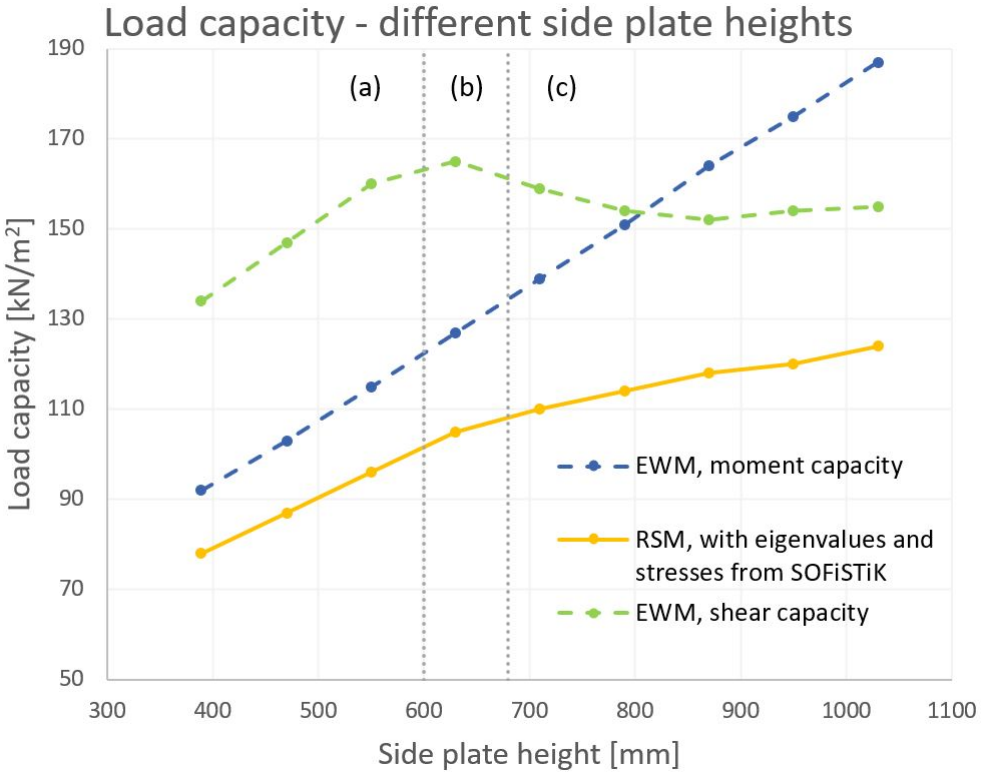


Figure 5.18: Load capacity of the box girder bridge calculated with the EWM and the RSM for different height of the side plates.

A comparison between the slenderness has been made for this study as well to be able to identify any difference in the methods there. The result is plotted and presented in figure 5.19. The EWM received two different slenderness curves. Slenderness related to plate like buckling of the equivalent plate (top plate and stiffeners) is governing at the original side plate height (398.2 mm) and at 470 mm. All side plates higher than that has a governing slenderness related to plate like buckling of the side plate.

The RSM curve follows a bi linear relation that is very close to the inclination of the two EWM curves. The inclination change occurs at about one side plate height later than the intersection of the two EWM curves. This inclination change happens to be where the buckling mode changes from a to b which was the case with the

capacity curve as well. This indicates that the change from buckling mode a to b is quite critical for the RSM calculations. The EWM does not have the same relation between capacity and slenderness but the buckling mode seems to affect more in this case. Note that the result from the buckling analysis is only used in the RSM calculations but the behaviour should apply for the EWM as well and the relation between them gets even more interesting because of this fact.

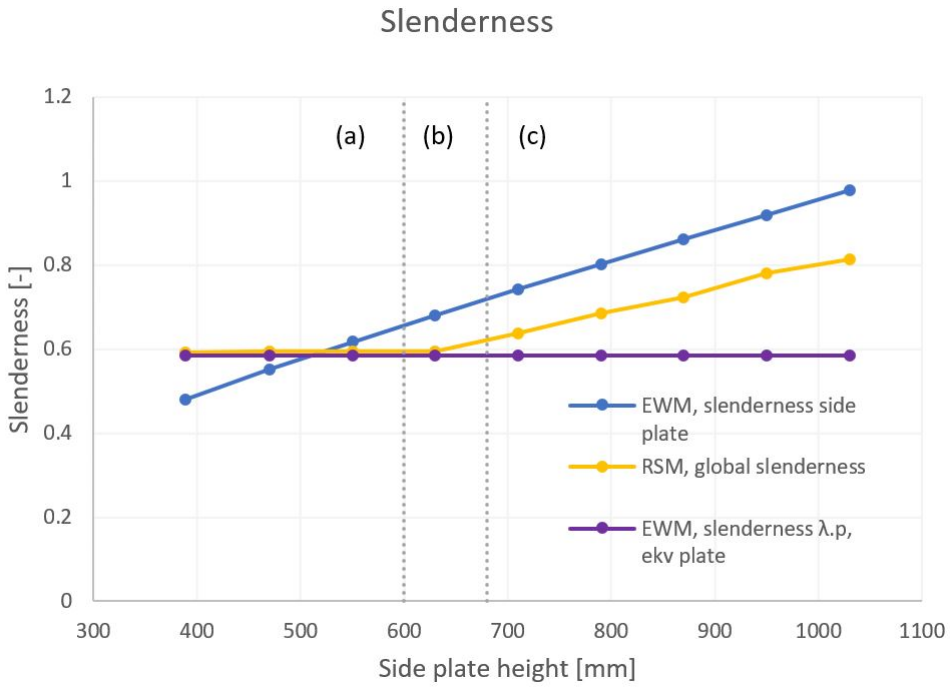
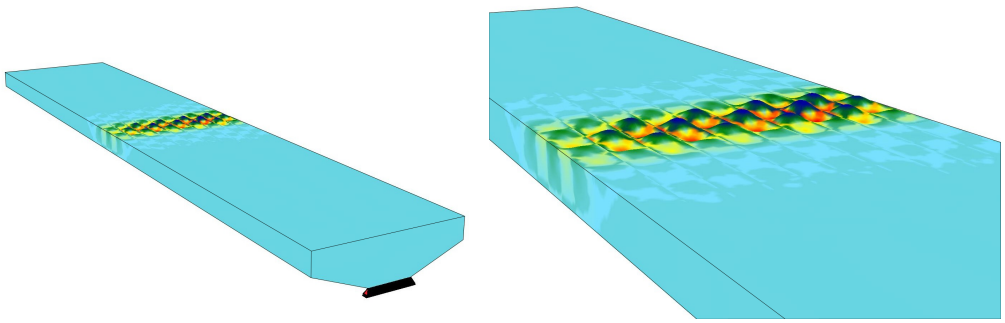


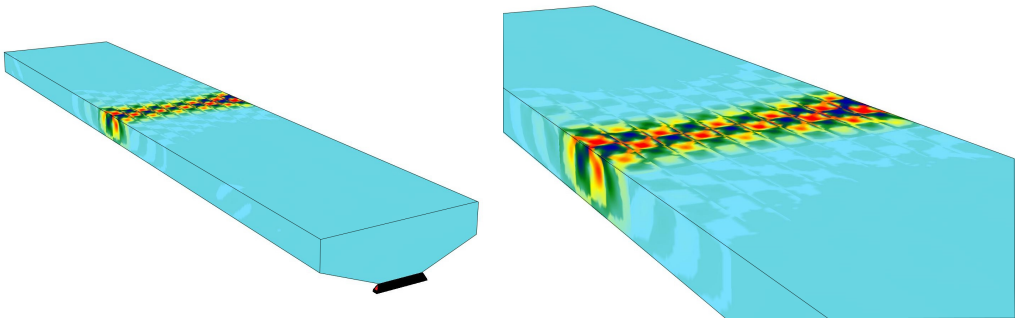
Figure 5.19: Slenderness with different side plate heights.

5.2.4.2 Buckling analysis

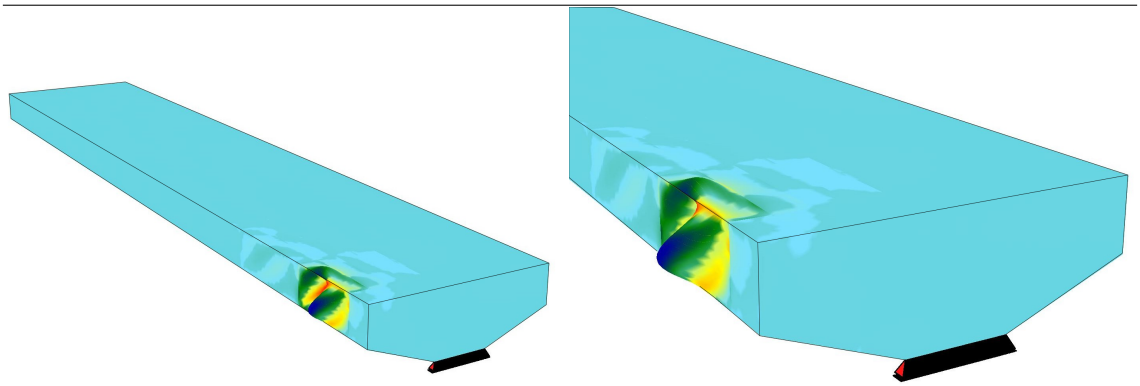
The buckling analysis resulted, as earlier mentioned, in four different buckling modes which are presented in figure 5.20 and table 5.7 below. The first buckling mode, mode a, is local normal stress buckling of the top plate between stiffeners in the middle of the span. This buckling mode is known by now since it occurs in the original design which is a part of all parametric studies. It's governing at the side plate height 389.2, 470 and 550 mm. The second mode, mode b, the same as mod a but in combination with local normal stress buckling of the side plate in the middle of the span. This mode occurs at a side plate height of 630 mm solely. Buckling mode c (at 710 and 790 mm) and d (above 790 mm) is one and two shear buckling modes respectively. Even though the two last modes are different modes there are not much of a difference between them. They occur at the same location as well as element and both are in fact shear buckling but since the EWM capacity curve seemed to change direction in this area they are kept separated.



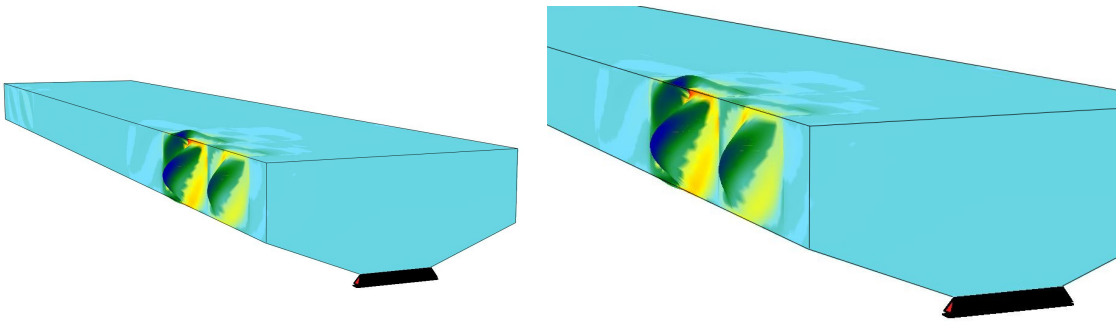
(a) Buckling mode a



(b) Buckling mode b



(c) Buckling mode c



(d) Buckling mode d

Figure 5.20: Three different buckling modes of the bridge with varying side plate height.

Buckling mode	Buckling behaviour	Thickness [mm]
(a)	Local normal buckling of the top plate.	389.2, 470, 550
(b)	Local normal stress buckling of the top- and side plate.	630
(c)	Shear buckling of the side plate.	710, 790
(d)	Shear buckling of the side plate in two end sections.	870, 950, 1030

Table 5.7: Buckling modes with varying side plate thickness.

The capacity in the RSM seems to be affected as soon as the side plate is involved in the buckling analysis since that's the major difference between buckling mode a and b. The change from normal stress buckling to shear buckling does not seem to change the resulting RSM capacity significantly. The same goes for the moment capacity in the EWM which does not seem to be affected by any buckling mode. Another situation is identified in the shear capacity with the EWM. The capacity increase is affected by the entrance of side plate buckling but the capacity decrease occurs when shear buckling is governing the structure. But the decrease wears off, and even changes to an increase as one extra mode is added. It's quite reasonable that the capacity can decrease when the structure changes element, location and type of the governing buckling mode. But that the capacity increase again when the same buckling mode spreads out more is perhaps not an intuitive guess. Maybe the post buckling reserve increases as the possible tension bands increases in numbers. In that case, the capacity won't increase further if not a third buckling mode will appear and increase the post buckling capacity even more. The increase wont be very high and the design not particularly material effective though. This is not very important to the comparison since both methods probably will behave the same with those high side plats. The most interesting part of this investigation is how the two methods cope with different buckling modes and overlaps from moment capacity as discussed earlier.

5.2.5 Distance between vertical stiffeners

Last but not least the capacity with a bridge with changed vertical stiffener distance is studied. The original distance is two metres which results in 8 equally spaced stiffeners along the span. The study consists of distances from 1.38 to 3.6 meters which means 12 to 5 equally spaced stiffeners respectively. One increase in space between the stiffeners means one less vertical stiffener along the span. This will result in a wider and wider span for the top plate and it's connecting longitudinal stiffeners.

5.2.5.1 Capacity comparison

The comparison between the two methods in the vertical stiffener study is presented in fig 5.21 below. The boundaries for the buckling modes are added as for the other studies. Three buckling modes occurred for this study where the first two, a and b, alternated for the shorter distances and a third, mode c, where governing for the wider spans. The buckling modes will be further discussed in the next section. The EWM calculation resulted in a moment capacity curve as the shear capacity where never governing. The EWM result were extended below 1.35 m and above 3.6 m as the calculations where not as tedious as the FE-computational time in the RSM calculations. The RSM result however is lower bond as for every other comparison where moment capacity is governing in the EWM. The RSM curve seems to be more constant through the shorter stiffener spans than the EWM curve which is decreasing quite linearly through out all calculated distances. A sudden increase in the capacity occurs in the RSM at a stiffener distance of 2.25 m which is just before the change to buckling mode c. Through out the wider stiffener distances, within buckling mode c, the RSM capacity starts to decrease in about the same manor as the EWM curve.

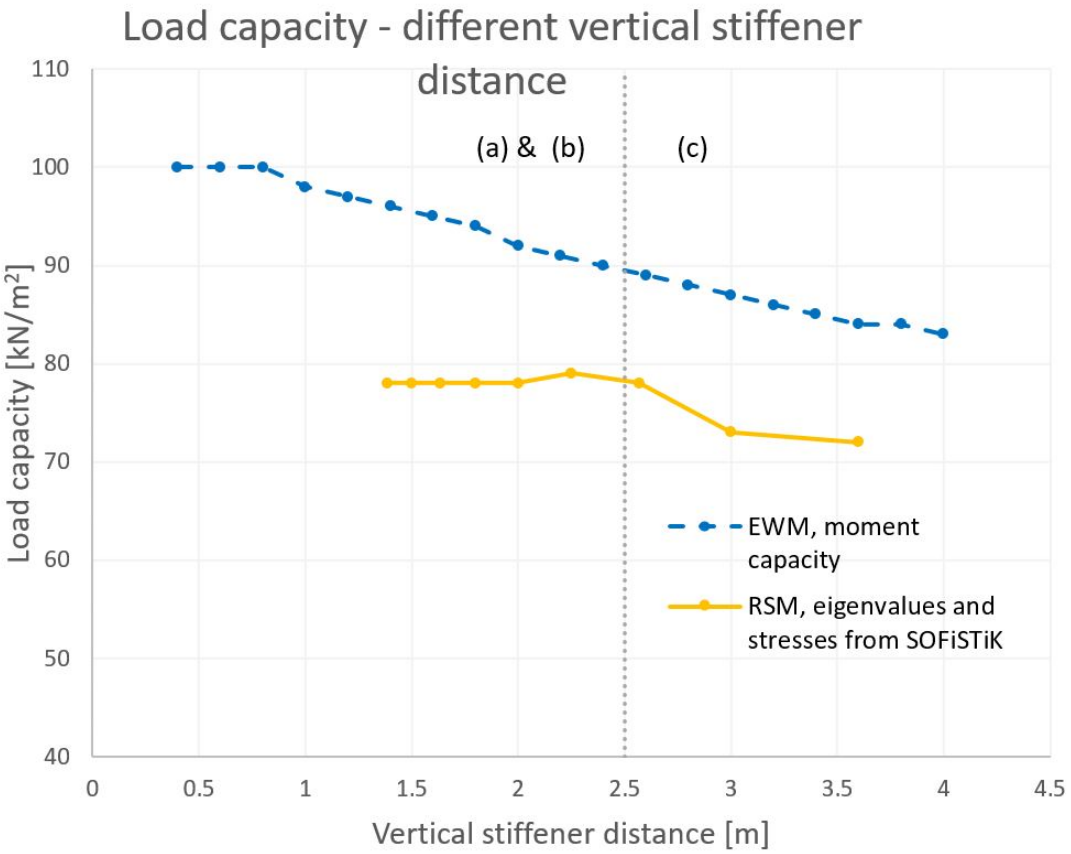


Figure 5.21: Load capacity with EWM and RSM with eigenvalues and stresses taken from Teddy for different vertical stiffener distance.

There are two odd things about the RSM capacity, the first constant part of the curve and the sudden increase by the 2.25 m distance calculation. A decreasing

slope like the EWM capacity seems more reasonable as an increasing span should lower the capacity. Hopefully an answer to this can be found in the slenderness curves or the buckling analysis.

The result from the slenderness calculations are presented in figure 5.22 below. The EWM has three governing slenderness curves, two of them which are almost the same and a third which is governing for the lower distances. The two very similar curves are slenderness for both plate like- and column like buckling of the equivalent plate, in orange and green respectively. The third slenderness is plotted in blue and corresponds to plate like buckling of the stiffener solely. The EWM capacity is rather unaffected by the fact that the slenderness changes from a constant to a rapidly increasing curve but then again the slenderness does not always correlate very well with the capacity. The RSM slenderness curve on the other hand has a shape that is almost an exact upside down reflection of the capacity curve. The slenderness is constant until 2.25 m stiffener distance where it drops down a bit to then increase. The drop at 2.25 is not as big as the increase in the capacity in the same point but it's not a one to one relation so a small slenderness change can result in a bigger capacity increase. But it's still not very clear why there is a slenderness drop at that distance nor why the slenderness is constant before that. The question will be saved to the buckling analysis but it can be concluded that the slenderness curves of EWM does not match the capacity as good as for the RSM. The Slenderness curves match each other's shape quite well however, besides where the slope change occurs.

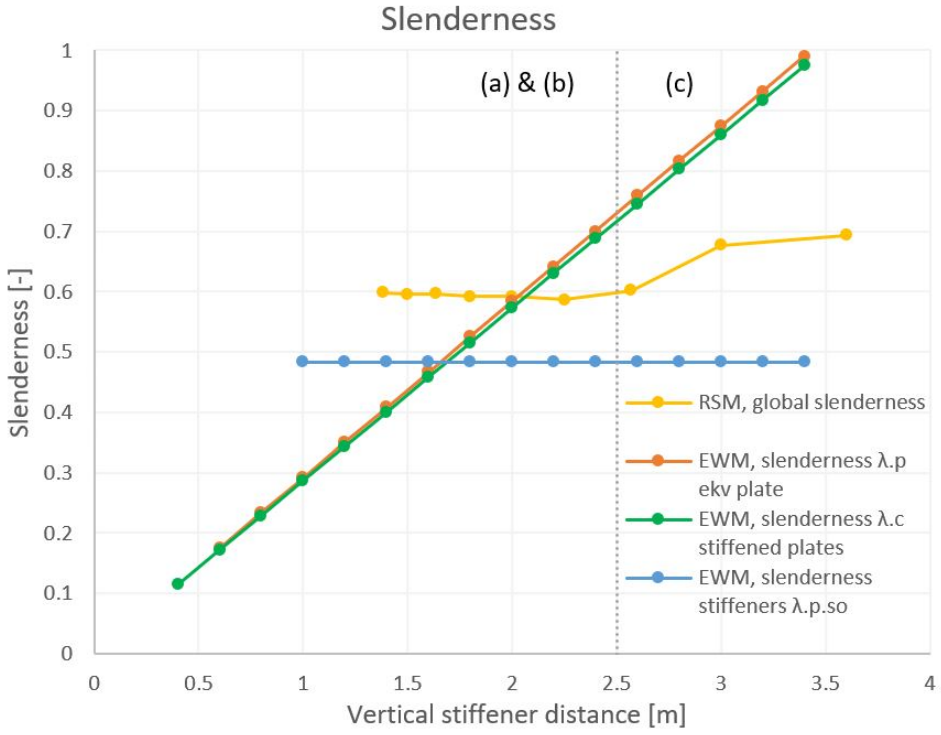
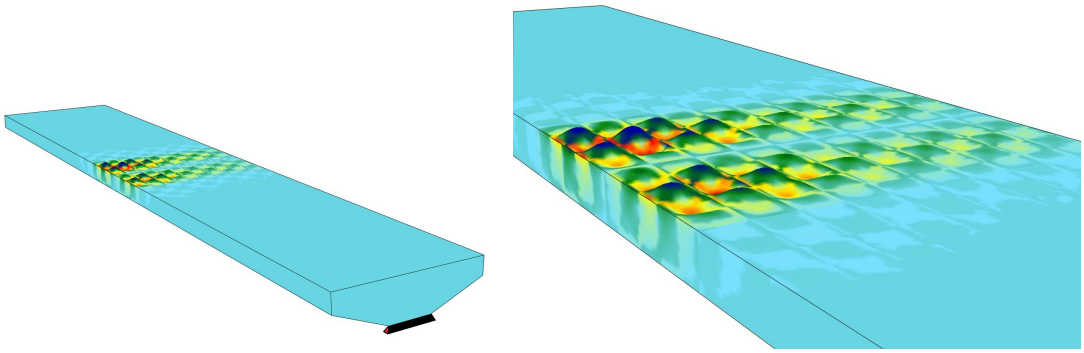


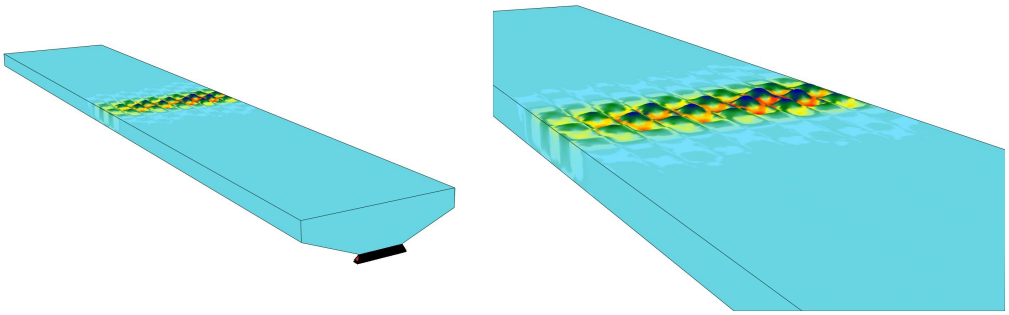
Figure 5.22: Slenderness with different vertical stiffener distance.

5.2.5.2 Buckling analysis

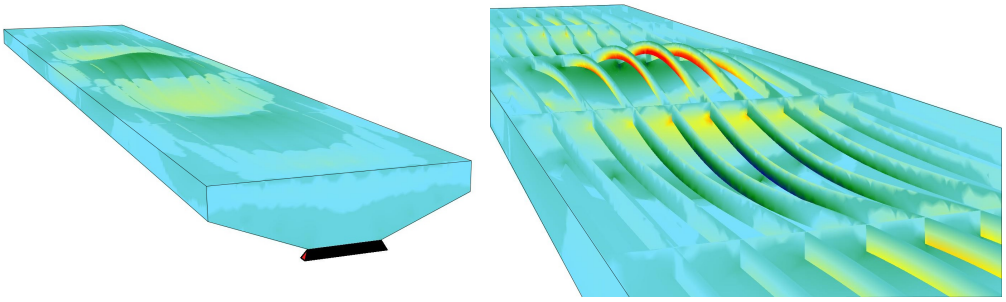
As earlier mentioned, the buckling analysis resulted in three different buckling modes which are presented in figure 5.23 and table 5.8 below. The two first ones are both local normal stress buckling of the top plate but mode a occurs when there is a transverse stiffener in the middle of the span so that it buckles on both sides. Buckling mode b is of course when there are an even amount of stiffeners so that it only buckles in one section of the beam. Mode c is a global normal stress buckling of the top plate along with the longitudinal stiffeners. Mode a and b occurs up to a 2.25 m stiffener distance and mode c above that.



(a) Buckling mode a



(b) Buckling mode b



(c) Buckling mode c

Figure 5.23: Three different buckling modes of the bridge with varying distance between transverse stiffeners.

Buckling mode	Buckling behaviour	Distance[m]
(a)	Local normal buckling of the top plate in two mid sections.	1.5, 1.8, 2.25
(b)	Local normal buckling of the top plate.	1.38, 1.64, 2
(c)	Global normal stress buckling of the top plate.	2.57, 3, 3.6

Table 5.8: Buckling modes with distance between vertical stiffeners.

The only difference between mode a and b is the geometry and the difference does not seem to affect the capacity in any of the methods. The capacity drop in the RSM occurs when the buckling mode becomes global, as in mode c, but the EWM capacity stays unaffected by this to. The reason for the little peak in capacity in the RSM at a 2.25 m stiffener distance remains uncertain as the buckling mode don't seem to give an answer to that. It could just be a rounding error in the calculations since it's only a matter of one extra kN/m^2.

6

Conclusion

After comparing the two methods on a single plate, a stiffened I-beam and a box-girder some conclusions are to be drawn. The four questions in the objective along with the results and discussions will be the building blocks of this chapter. The questions are turned in to headers which divide the sections and a fifth section is added with suggestions for further studies.

6.1 Observed differences

The RSM is lower bond in almost all cases, there are a few exceptions through out the different studies. Firstly when the structure consists of one uni-axially stressed plate, as the single plate study. The methods are in fact mathematically the same so the capacity are as well. The methods was about equally time consuming so there is actually no difference between the methods in this case.

Another case where the EWM is not higher bond is where interaction between moment- and shear capacity is governing within the EWM. This was only the case in the I-beam so this hypothesis has not been tested on other structures. The RSM was though higher bond when this was the case but as mentioned before the assumption that interaction being needed in the I-beam could have been too conservative.

A more certain conclusion is that the methods are close when shear capacity is governing in the EWM. In on case, the side plate thickness study, the RSM gives even higher capacity than EWM. So there seems to be potential for the RSM in designs where the shear capacity is governing.

6.2 Pros and cons

The RSM proved to result in a high capacity which of course is a major advantage when it comes to design where a more material efficient structure is obtained. This with a fully analytical calculation procedure without any involvement of FEM. The method where equally, or in some cases less, time consuming compared to the RSM for the studied cases within this thesis. The general opinion from the state of the art literature study where in fact the opposite which is quite interesting. It should

be mentioned that the interpretation of FEM with the RSM played a big role in the time consumption of the calculations. The analytical part where experienced as more troublesome than whats revealed in a first glance in Eurocode so it's not just the FEM part that takes time. A disadvantage with the RSM is however it's inability to be combined with FEM which where not a big problem for the studied cases but can definitely be for larger and more complex structures. Also when many load cases should be calculated for a structure which is the normal case in a real bridge design this makes it unnecessarily time consuming.

The obvious advantage with RSM is in stead the possibility to combine it with FEM for the reasons mentioned above. If the method is combined with FEM the structures behaviour is better visialized than a solely analytial calculation which results in a gained understanding for the designer which is always good. The ability to analyze multi actually loaded structures is another thing that is exclusive for the RSM. This is not further studied in this thesis but mentioned in the literature review. The often lower bond solution makes this method less attractive compared to the EWM which of course is a disadvantage. The method is quite general and shortly explained in Eurocode which makes it hard to adapt to a specific structure if the designer is not experienced with the method. These disadvantages may be the reason why the Swedish road administration advises not to use the method in the Swedish annex to Eurocode.

6.3 FEM-implementation and the parametric tool

When combining RSM with FEM capacity is most often gained compared to the fully analytical solution which is of course great. The capacity is however not gained enough to compete with the EWM in most cases as this study has proven. The extra time spent on building a FEM model, running the analysis and validating the results may therefore not be worth it if the final capacity is lower than that of the EWM. But for cases where the FEM calculation is favourable in the time aspect as mentioned earlier the lower capacity may be worth it. The parametric tool built up in Grasshopper can actually be crucial in this case since changes to the design becomes very smooth compared to rebuilding the model in the FEM software. The experience in this study where however that the time saved by the parametric tool where lost in the very tedious FEM analysis which for the box section took over 40 minutes with the sufficient mesh. So the conclusion is that both FEM implementation and a parametric tool can be effective in combination with the RSM but weren't for the studied cases in this thesis.

6.4 Recommendations for further studies

Throughout this study a lot of knowledge and understanding where gained in the calculation methods for steel structures in cross section class 4 but some questions where left unanswered or at least not fully answered. Some suggestions for further studies that may clear this out are presented below.

Since RSM is the only method that can be used for multi axially loaded structures in Eurocode it could be interesting to do a similar parametric study on such structures and compare to the procedure suggested in [Gren et al., 2019] to do so with the EWM. The design should have a big impact in this cases and the comparison can be more complex since the load cases can be alternated as well as the designs.

Time consumption aspect did not really come in to play in this thesis so it should be interesting to study cases where it may. Such cases could be larger and more complex structures with a lot of sub plates. In those cases the EWM can be tedious. Another case could be similar structures but with a lot of load cases which makes the EWM somewhat more troublesome. The load cases could include both positive and negative bending moments so that different parts are in compression at different times.

A third and final suggestion is to look at continuously supported structures where large bending moments and shear forces can occur in the same area e.x. close to supports. The I-beam study indicated that the methods where close when interaction between these two occurred so a further investigation to this would be interesting.

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A

Singel plate calculation

Plate calculation

Geometry

$h_p := 3m \quad b_p := 2m \quad t_p := 20mm$

$A := b_p \cdot t_p = 0.04\,m^2$

Material parameters

$f_y := 235MPa \quad E := 210GPa \quad \nu := 0.3$

$\gamma_{M0} := 1 \quad \gamma_{M1} := 1$

Loads

$q := 324\,\frac{kN}{m}$

$N_{Ed} := q \cdot b_p = 648 \cdot kN$

Cross section class determination

$\varepsilon := \sqrt{\frac{235MPa}{f_y}} = 1 \quad c_p := b_p$

$\beta := \frac{c_p}{t_p} = 100 \quad 42 \cdot \varepsilon = 42$

$\beta \le 42 \cdot \varepsilon = 0$ Cross section class 4

Effective width method

$\psi := 1 \quad k_\sigma := 4$

$\lambda_p := \frac{\beta}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = 1.7606$

$\rho := \frac{\lambda_p - 0.055 \cdot (3 + \psi)}{\lambda_p^2} = 0.497$

$b_{eff} := \rho \cdot b_p = 0.994\,m$

$A_{eff} := b_{eff} \cdot t_p = 0.0199\,m^2$

$N_{c.Rd} := A_{eff} \cdot \frac{f_y}{\gamma_{M0}} = 4.672 \times 10^3 \cdot kN$

Critical column-like buckling stress

$a := h_p \quad b := b_p$

$\sigma_{cr.c} := \frac{\pi^2 \cdot E \cdot t_p^2}{12 \cdot (1 - \nu^2) \cdot a^2} = 8.4356 \cdot MPa$

Critical Plate-like buckling stress

$\sigma_E := \frac{\pi^2 \cdot E \cdot t_p^2}{12 \cdot (1 - \nu^2) \cdot b^2} = 18.98 \cdot MPa$

$\sigma_{cr.p} := k_\sigma \cdot \sigma_E = 75.92 \cdot MPa$

Interaction

$\xi := \frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1 = 8$

$\xi := \begin{cases} 1 & \text{if } \xi > 1 \\ \xi & \text{otherwise} \end{cases} = 1$

Reduction for collumn-like buckling

$I_z := \frac{b_p^3 \cdot t_p}{12} = 0.0133\,m^4$

$L_{cr} := h_p$

$N_{cr} := \frac{(\pi)^2 \cdot E \cdot I_z}{L_{cr}^2} \cdot \frac{1}{1 - \nu^2} = 3.3742 \times 10^6 \cdot kN$

$P_{cr} := \sigma_{cr.p} \cdot b \cdot t_p = 3.0368 \times 10^3 \cdot kN$

$\lambda := \sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}} = 0.0372$

$\alpha := 0.21$ Buckling curve a

$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.4836$

$\chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = 1.0354$

$\chi := \begin{cases} \chi & \text{if } \chi \le 1 \\ 1 & \text{if } \chi > 1 \end{cases} = 1$

$\rho_c := (\rho - \chi) \cdot \xi \cdot (2 - \xi) + \chi = 0.497$

$\rho := \begin{cases} \rho_c & \text{if } \rho \ne \rho_c \\ 1 & \text{if } \rho_c > 1 \\ \rho & \text{otherwise} \end{cases} = 0.497$

$b_{eff} := \rho \cdot b_p = 0.994\,m$

$A_{eff} := b_{eff} \cdot t_p = 0.0199\,m^2$

$N_{b.Rd} := \frac{\chi \cdot A_{eff} \cdot f_y}{\gamma_{M0}} = 4.672 \times 10^3 \cdot kN$

$q_r := \frac{N_{b.Rd}}{b_p} = 2.336 \times 10^3 \cdot \frac{kN}{m}$ Capacity with RSM

Reduced stress method

Assumed capacity

$q_r := 2337\,\frac{kN}{m}$

Governing stresses

$\sigma_{x.Ed} := \frac{q}{t_p} = 116.85 \cdot MPa \quad \sigma_{y.Ed} := 0 \quad \tau_{Ed} := 0$

Critical stresses (Same as EWM)

$\sigma_{cr.x} := \max(\sigma_{cr.p}, \sigma_{cr.c}) = 75.92 \cdot MPa \quad \sigma_{cr} := \sigma_{cr.x}$

Amplification fators

$\alpha_{cr.x} := \frac{\sigma_{cr.x}}{\sigma_{x.Ed}} = 0.6497 \quad \alpha_{ult.k} := \frac{f_y}{\sigma_{x.Ed}} = 2.0111$

Slenderness factor

$\lambda_{p.RSM} := \sqrt{\frac{\alpha_{ult.k}}{\alpha_{cr.x}}} = 1.7594$

Reduction factor for plate like buckling

$\rho_x := \frac{\lambda_{p.RSM} - 0.055 \cdot (3 + \psi)}{\lambda_{p.RSM}^2} = 0.4973$

Reduction factor for collumn like buckling

$\phi_{RSM} := 0.5 \cdot \left[1 + \alpha \cdot \left(\lambda_{p.RSM} - 0.2\right) + \lambda_{p.RSM}^2\right] = 2.2114$

$\chi_c := \frac{1}{\phi_{RSM} + \sqrt{\phi_{RSM}^2 - \lambda_{p.RSM}^2}} = 0.2816$

Interaction

$\rho_{ov} := (\rho_x - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.4973$

Capacity check

$\left(\frac{\sigma_{x.Ed}}{\frac{\rho_c \cdot f_y}{\gamma_{M1}}} \right)^2 = 0.9997$

Ok! q=2337 is the final capacity

B

I-beam

B.1 Changed stiffener width

LIX EWM
LXIX RSM

I-beam - EWM 50mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 50mm$ $b_{2.st} := 50mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 544 \cdot cm^2$

Load:

$Q_d := 152 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.359 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.032 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.513$ SS-EN 1993-1-3 5.2(5.2) p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.333 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1) Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.809$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\,kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.809$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.648$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.743$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.844$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.779$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.692$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.888$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.941$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.638$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.779$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.779$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.16 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

Interaction not needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

Interaction not needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} = \text{"Interaction needed"} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.364 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 0.996$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.941$ EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0 \qquad \eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.262$$

I-beam - EWM 50mm Shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 50mm$ $b_{2.st} := 50mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 544 \cdot cm^2$

Load:

$Q_d := 161 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.439 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.153 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations SS-EN 1993-1-3 5.2(5.2)

$\frac{M_{Ed}}{M_{f.Rd}} = 1.603$ p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.708 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.857$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3 kN$

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.857$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.686$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.787$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.894$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.825$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.733$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.94$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.997$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.676$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.825$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.825$

$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.997$ EC1993-1-5 Ch. 7.1(5)

V.bw.Rd, EWM with dirrerent stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.16 \times 10^3 \cdot kN \cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^4 kN \cdot m$ From Typ 1, rambolls EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^4 \ kN \cdot m$ From Typ 1 EWM Ramboll

$M_{pl.Rd.100} := 1.389 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^4 \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^4 kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^4 kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^4 \ kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.364 \times 10^4 \cdot kN \cdot m$$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 1.055$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^3 \cdot kN \cdot m$ From Typ 1, rambolls EWM beräkningar

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \le 1 = 0$

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 1.392$

$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$

I-beam - EWM 50mm Interaction

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 50mm$ $b_{2.st} := 50mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 544 \cdot cm^2$

Load:

$Q_d := 131 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.171 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 1.752 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

SS-EN 1993-1-3 5.2(5.2)

$\frac{M_{Ed}}{M_{f.Rd}} = 1.304$

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 5.458 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The contributon from the flange EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$

EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.697$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.697$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.558$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.64$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.728$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.671$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.596$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.765$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.811$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.55$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.671$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.671$

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.16 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} = \text{"Interaction needed"} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From Ramboll EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.364 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.859$$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From Ramboll EWM calculations

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.811$$ EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \leq 1 = 1$$

$$\eta_1 \geq \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 0.991$$

I-beam - EWM 100mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 100mm$ $b_{2.st} := 100mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 568 \cdot cm^2$

Load:

$Q_d := 155 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.386 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.073 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.543$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.458 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.825$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.825$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6\,N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.66$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.757$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.861$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.794$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.706$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.905$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.959$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.651$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.794$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.794$

V.bw.Rd, EWM with dirrerent stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.289 \times 10^{3} \cdot kN \cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$ From Ramboll EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \ kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.389 \times 10^{4} \cdot kN \cdot m$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 0.998$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$ From Ramboll EWM calculations

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.905$ EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0 \qquad \eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.23$$

I-beam - EWM 100mm Shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 100mm$ $b_{2.st} := 100mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 568 \cdot cm^2$

Load:

$Q_d := 171 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.529 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.286 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.703$ SS-EN 1993-1-3 5.2(5.2) p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 7.125 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1) Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.91$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3 kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.91$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.729$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.836$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.95$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.876$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.778$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.999$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.059$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.718$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.876$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.876$

V.bw.Rd, EWM with dirrerent stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.289 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.453$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$\text{Check} := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} = \text{"Interaction needed"} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From Ramboll EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.389 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 1.101$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From Ramboll EWM calculations

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.999$ EC1993-1-5 Ch. 7.1(5)

I-beam - EWM 100mm Interaction

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 100mm$ $b_{2.st} := 100mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 568 \cdot cm^2$

Load:

$Q_d := 136 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.216 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 1.818 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.354$ p.86 BK

SS-EN 1993-1-3 5.2(5.2)

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 5.667 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.724$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.724$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.579$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.665$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.755$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.697$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.619$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.794$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.842$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.571$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.697$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.697$

V.bw.Rd, EWM with dirrerent stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.289 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$$M_{Ed} < M_{f.Rd} = 0$$

$$V_{Ed} < V_{bw.RD} = 1$$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$\text{Check} := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From Ramboll EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.389 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.875$$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From Ramboll EWM calculations

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.794$$ EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 1$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 0.998$$

I-beam - EWM 150mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 150mm$ $b_{2.st} := 150mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 592 \cdot cm^2$

Load:

$Q_d := 157 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.403 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.099 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.563$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.542 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.835$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3 kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.835$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.669$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.767$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.872$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.805$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.715$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.917$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.972$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.659$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.805$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.805$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.407 \times 10^{3} \cdot kN \cdot m$$

Interaction not needed because the load can be carried with the flanges contribution and the following conditions are met.

$$M_{Ed} < M_{f.Rd} = 0$$

$$V_{Ed} < V_{bw.RD} = 1$$

Interaction not needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$$

Ramboll EWM calculations

$$M_{pl.Rd.50} := 1.364 \times 10^{4} \, kN \cdot m$$

$$M_{pl.Rd.100} := 1.389 \times 10^{4} \, kN \cdot m$$

$$M_{pl.Rd.150} := 1.407 \times 10^{4} \, kN \cdot m$$

$$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$$

$$M_{pl.Rd.250} := 1.429 \times 10^{4} \, kN \cdot m$$

$$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$$

$$M_{pl.Rd.350} := 1.448 \times 10^{4} \, kN \cdot m$$

$$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$$

$$M_{pl.Rd.500} := 1.465 \times 10^{4} \, kN \cdot m$$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.407 \times 10^{4} \cdot kN \cdot m$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.997$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$$

Ramboll EWM calculations

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.872$$

EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \leq 1 = 0$$

$$\eta_1 \geq \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.198$$

I-beam - EWM 150mm Shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 150mm$ $b_{2.st} := 150mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 592 \cdot cm^2$

Load:

$Q_d := 180 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.609 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.407 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.792$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 7.5 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.958$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3 kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.958$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.767$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.88$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 1$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.922$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.819$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 1.051$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.114$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.755$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.922$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.922$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.407 \times 10^{3} \cdot kN \cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$$M_{Ed} < M_{f.Rd} = 0$$

$$V_{Ed} < V_{bw.RD} = 1$$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} = \text{"Interaction needed"} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$$

From EWM calculations Ramboll

$$M_{pl.Rd.50} := 1.364 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.100} := 1.389 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.150} := 1.407 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$$

$$M_{pl.Rd.250} := 1.429 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$$

$$M_{pl.Rd.350} := 1.448 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$$

$$M_{pl.Rd.500} := 1.465 \times 10^{4} \ kN \cdot m$$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.407 \times 10^{4} \cdot kN \cdot m$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 1.144$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$$

From EWM calculations Ramboll

EC1993-1-5 Ch. 7.1(5)

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \le 1 = 0$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 1.505$$

I-beam - EWM 150mm Interaction capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 150mm$ $b_{2.st} := 150mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 592 \cdot cm^2$

Load:

$Q_d := 139 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.243 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 1.859 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

SS-EN 1993-1-3 5.2(5.2)

$\frac{M_{Ed}}{M_{f.Rd}} = 1.384$ p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 5.792 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.74$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.74$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.592$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.679$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.772$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.712$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.633$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.812$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.86$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.583$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.712$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.712$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.407 \times 10^{3}\cdot\text{kN}\cdot\text{m}$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right)\cdot\left(2\cdot\eta_3 - 1\right)^2 \le 1 = 1$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right)\cdot\left(2\cdot\eta_3 - 1\right)^2 = 0.99$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$\text{Check} := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} = \text{"Interaction needed"} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5\cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419\cdot 10^{4}\,\text{kN}\cdot\text{m}$ Ramboll EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^{4}\,\text{kN}\cdot\text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4}\,\text{kN}\cdot\text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4}\cdot\text{kN}\cdot\text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4}\,\text{kN}\cdot\text{m}$

$M_{pl.Rd.300} := 1.430\cdot 10^{4}\,\text{kN}\cdot\text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4}\,\text{kN}\cdot\text{m}$

$M_{pl.Rd.400} := 1.454\cdot 10^{4}\,\text{kN}\cdot\text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4}\,\text{kN}\cdot\text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.407 \times 10^{4}\cdot\text{kN}\cdot\text{m}$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 0.883$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3}\cdot\text{kN}\cdot\text{m}$ From EWM calculations Ramboll

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.772$ EC1993-1-5 Ch. 7.1(5)

I-beam - EWM 200mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 200mm$ $b_{2.st} := 200mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 616 \cdot cm^2$

Load:

$Q_d := 158 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.412 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.113 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.573$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.583 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

Flänsarnas bidrag enligt EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.841$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.841$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.673$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.772$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.878$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.81$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.719$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.923$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.978$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.663$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.81$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.81$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.513 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

Interaction not needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 0.995$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From Ramboll EWM calculations

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.841$ EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.166$$

I-beam - EWM 200mm shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 200mm$ $b_{2.st} := 200mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 616 \cdot cm^2$

Load:

$Q_d := 187 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.672 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.5 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.862$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 7.792 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.995$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3 kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.995$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.797$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.914$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 1.039$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.958$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.851$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 1.092$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.158$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.785$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.958$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.958$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.513 \times 10^{3} \cdot kN \cdot m$$

Interaction is not needed because the load can be carried with the flanges contribution and the following conditions are met.

$$M_{Ed} < M_{f.Rd} = 0$$

$$V_{Ed} < V_{bw.RD} = 1$$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$$

From EWM calculations Ramboll

$$M_{pl.Rd.50} := 1.364 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.100} := 1.389 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.150} := 1.407 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$$

$$M_{pl.Rd.250} := 1.429 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$$

$$M_{pl.Rd.350} := 1.448 \times 10^{4} \ kN \cdot m$$

$$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$$

$$M_{pl.Rd.500} := 1.465 \times 10^{4} \ kN \cdot m$$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.419 \times 10^{4} \cdot kN \cdot m$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 1.178$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$$

From EWM calculations Ramboll

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.995$$

EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.538$$

I-beam - EWM 200mm Interaction

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 200mm$ $b_{2.st} := 200mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 616 \cdot cm^2$

Load:

$Q_d := 142 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.269 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 1.899 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.414$ SS-EN 1993-1-3 5.2(5.2) p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 5.917 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1) Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.756$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3 kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.756$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.605$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.694$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.789$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.728$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.646$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.829$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.879$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.596$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.728$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.728$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.513 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \le 1 = 1$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 0.99$$

Interaction not needed because the load can be carried with the flanges contribution and the following conditions are met.

$$M_{Ed} < M_{f.Rd} = 0$$

$$V_{Ed} < V_{bw.RD} = 1$$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$\text{Check} := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$$

From EWM calculations Ramboll

$$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$$

From EWM calculations Ramboll

$$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.895$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

From Typ 1, rambolls EWM calculations

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.756$$

EC1993-1-5 Ch. 7.1(5)

I-beam - EWM 250mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 250mm$ $b_{2.st} := 250mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 640 \cdot cm^2$

Load:

$Q_d := 159 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.421 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.126 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.583$ SS-EN 1993-1-3 5.2(5.2)

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The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.625 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.846$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.846$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6\,N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.677$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.777$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.883$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.815$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.724$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.929$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.984$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.667$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.815$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.815$

V.bw.Rd, EWM with dirrerent stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.609 \times 10^{3} \cdot kN \cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} = \text{"Interaction needed"} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$ The EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} kN \cdot m$

$M_{pl.Rd.100} := 1.389 \times 10^{4} kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^{4} kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^{4} kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^{4} kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^{4} kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.429 \times 10^{4} \cdot kN \cdot m$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.995$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$

Ramboll EWM calculations

EC1993-1-5 Ch. 7.1(5)

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.815$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.142$$

I-beam - EWM 250mm shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 250mm$ $b_{2.st} := 250mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 640 \cdot cm^2$

Load:

$Q_d := 195 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.743 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.607 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.942$ SS-EN 1993-1-3 5.2(5.2)

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The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 8.125 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 1.038$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 1.038$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6\ N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.831$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.953$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 1.083$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.999$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.888$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 1.139$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.207$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.818$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.999$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.999$

V.bw.Rd, EWM with dirrerent stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.609 \times 10^{3} \cdot kN \cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$ Ramboll EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \ kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \ kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.429 \times 10^{4} \cdot kN \cdot m$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 1.22$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$ Ramboll EWM calculations

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.999$ EC1993-1-5 Ch. 7.1(5)

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \leq 1 = 0$

$\eta_1 \geq \dfrac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 1.591$

I-beam - EWM 250mm interaction

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 250mm$ $b_{2.st} := 250mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 640 \cdot cm^2$

Load:

$Q_d := 145 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.296 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 1.939 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.444$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.042 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.772$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.772$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.618$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.709$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.805$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.743$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.66$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.847$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.898$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.609$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.743$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.743$

V.bw.Rd, EWM with dirrerent stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.609 \times 10^{3} \cdot kN \cdot m$$

Interaction is not needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$ Ramboll EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^{4} kN \cdot m$

$M_{pl.Rd.100} := 1.389 \times 10^{4} kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^{4} kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^{4} kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^{4} kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^{4} kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.429 \times 10^{4} \cdot kN \cdot m$$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.907$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$ Ramboll EWM calculations

$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.743$ EC1993-1-5 Ch. 7.1(5)

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \leq 1 = 1$

$\eta_1 \geq \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 0.995$

I-beam - EWM 300mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 300mm$ $b_{2.st} := 300mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 664 \cdot cm^2$

Load:

$Q_d := 160 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.43 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.139 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\xi(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.593$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.667 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

Flänsarnas bidrag enligt EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.851$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3 kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.851$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.682$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.782$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.889$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.82$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.728$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.935$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.99$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.671$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.82$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.82$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.736 \times 10^{3} \cdot kN \cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$ From Rambolls EWM calculations

$M_{pl.Rd.50} := 1.364 \times 10^{4} kN \cdot m$

$M_{pl.Rd.100} := 1.389 \times 10^{4} kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^{4} kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^{4} kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^{4} kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^{4} kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.43 \times 10^{4} \cdot kN \cdot m$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 1$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

From rambolls EWM calculations

$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$

EC1993-1-5 Ch. 7.1(5)

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.782$

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0$

$\eta_1 \ge \dfrac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.119$

I-beam - EWM 300mm Shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 300mm$ $b_{2.st} := 300mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 664 \cdot cm^2$$

Load:

$$Q_d := 204 \frac{kN}{m}$$

$$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.824 \times 10^4 \cdot kN \cdot m$$

$$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.728 \times 10^3 \cdot kN$$

Material

$\gamma_M := 1$ $E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 2.031$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 8.5 \cdot MPa$$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Design resistance for shear:

$\eta := 1$

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 1.085$$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 1.085$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6\ N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.869$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.997$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 1.133$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.045$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.929$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 1.192$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.263$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.856$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.045$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.045$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.736 \times 10^3 \cdot kN \cdot m$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 1.643$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^4 kN \cdot m$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.100} := 1.389 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^4 \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^4 kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^4 \ kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^4 kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^4 \ kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.43 \times 10^4 \cdot kN \cdot m$$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 1.275$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^3 \cdot kN \cdot m$ From EWM calculations Ramboll

EC1993-1-5 Ch. 7.1(5)

$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.997$

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \leq 1 = 0$

$\eta_1 \geq \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$

I-beam - EWM 300mm interaction

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 300mm$ $b_{2.st} := 300mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 664 \cdot cm^2$

Load:

$Q_d := 148 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.323 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 1.979 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.474$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.167 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.787$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.787$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.631$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.723$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.822$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.758$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.674$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.865$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.916$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.621$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.758$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.758$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.736 \times 10^{3} \cdot kN \cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} = \text{"Interaction needed"} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} kN \cdot m$

$M_{pl.Rd.100} := 1.389 \times 10^{4} kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^{4} kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^{4} kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^{4} kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^{4} kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.43 \times 10^{4} \cdot kN \cdot m$$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.925$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$ From EWM calculations Ramboll

EC1993-1-5 Ch. 7.1(5)

$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.723$

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \le 1 = 1$

$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 0.999$

I-beam - EWM 350mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 350mm$ $b_{2.st} := 350mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 688 \cdot cm^2$

Load:

$Q_d := 162 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.448 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.166 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.613$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.75 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.862$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.862$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.69$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.792$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.9$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.83$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.738$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.946$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.003$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.68$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.83$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.83$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.937 \times 10^{3} \cdot kN \cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} kN \cdot m$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} kN \cdot m$

$M_{pl.Rd.100} := 1.389 \times 10^{4} kN \cdot m$

$M_{pl.Rd.150} := 1.407 \times 10^{4} kN \cdot m$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot kN \cdot m$

$M_{pl.Rd.250} := 1.429 \times 10^{4} kN \cdot m$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.350} := 1.448 \times 10^{4} kN \cdot m$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} kN \cdot m$

$M_{pl.Rd.500} := 1.465 \times 10^{4} kN \cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.448 \times 10^{4} \cdot kN \cdot m$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 1$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot kN \cdot m$ From EWM calculations Ramboll

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.738$ EC1993-1-5 Ch. 7.1(5)

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \le 1 = 0$

$\eta_1 \ge \dfrac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 1.086$

I-beam - EWM 350mm Shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 350mm$ $b_{2.st} := 350mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 688 \cdot cm^2$

Load:

$Q_d := 219 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.958 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.928 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 2.181$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 9.125 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 1.165$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 1.165$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.933$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 1.07$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 1.217$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.122$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.997$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 1.279$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.356$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.919$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.122$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.122$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.937 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$$M_{Ed} < M_{f.Rd} = 0$$

$$V_{Ed} < V_{bw.RD} = 1$$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$$

From EWM calculations Ramboll

$$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$$

From EWM calculations Ramboll

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.448 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 1.352$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

From EWM calculations Ramboll

$$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

EC1993-1-5 Ch. 7.1(5)

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.997$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.727$$

I-beam - EWM 350mm Interaction

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 350mm$ $b_{2.st} := 350mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 688 \cdot cm^2$

Load:

$Q_d := 152 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.359 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.032 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.513$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.333 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.809$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.809$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.648$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.743$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.844$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.779$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.692$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.888$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.941$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.638$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.779$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.779$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right)\cdot\left(2\cdot\eta_3 - 1\right)^2 = 0.994$$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50mm \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100mm \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150mm \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200mm \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250mm \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300mm \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350mm \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400mm \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 2.937\times 10^3\cdot kN\cdot m$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5\cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419\cdot 10^4 kN\cdot m$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364\times 10^4\ kN\cdot m$

$M_{pl.Rd.100} := 1.389\times 10^4\ kN\cdot m$

$M_{pl.Rd.150} := 1.407\times 10^4\ kN\cdot m$

$M_{pl.Rd.250} := 1.429\times 10^4\ kN\cdot m$

$M_{pl.Rd.300} := 1.430\cdot 10^4 kN\cdot m$

$M_{pl.Rd.350} := 1.448\times 10^4\ kN\cdot m$

$M_{pl.Rd.400} := 1.454\cdot 10^4 kN\cdot m$

$M_{pl.Rd.500} := 1.465\times 10^4\ kN\cdot m$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50mm \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100mm \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150mm \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200mm \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250mm \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300mm \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350mm \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400mm \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.448\times 10^4\cdot kN\cdot m$$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.938$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978\times 10^3\cdot kN\cdot m$ From EWM calculations Ramboll

EC1993-1-5 Ch. 7.1(5)

$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.692$

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right)\cdot\left(2\cdot\eta_3 - 1\right)^2 \le 1 = 1$

$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$

I-beam - EWM 400mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 400mm$ $b_{2.st} := 400mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 712 \cdot cm^2$

Load:

$Q_d := 162 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.448 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.166 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.613$ SS-EN 1993-1-3 5.2(5.2) p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.75 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1) Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.862$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.862$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.69$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.792$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.9$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.83$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.738$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.946$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.003$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.68$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.83$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.83$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 3.138 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.454 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.996$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$\eta_{3.} := \frac{V_{Ed}}{V_{bw.RD}} = 0.69$ EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.051$$

I-beam - EWM 400mm Shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 400mm$ $b_{2.st} := 400mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 712 \cdot cm^2$

Load:

$Q_d := 234 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 2.092 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 3.129 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 2.33$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 9.75 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 1.245$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 1.245$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6\,N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.997$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 1.144$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 1.3$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.199$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 1.065$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 1.367$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.449$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.982$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.199$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.199$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 3.138 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$\text{Check} := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} = \text{"Interaction needed"} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.454 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 1.439$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.997$ EC1993-1-5 Ch. 7.1(5)

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \le 1 = 0$

$\eta_1 \ge \dfrac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 1.817$

I-beam - EWM 400mm Interaction

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 400mm$ $b_{2.st} := 400mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 712 \cdot cm^2$

Load:

$Q_d := 155 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.386 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.073 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.543$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.458 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.825$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.825$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.66$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.757$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.861$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.794$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.706$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.905$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.959$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.651$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.794$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.794$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 3.138 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.454 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.953$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

EC1993-1-5 Ch. 7.1(5)

$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.66$

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \leq 1 = 1$

$\eta_1 \geq \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 0.992$

I-beam - EWM 500mm Moment capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 500mm$ $b_{2.st} := 500mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 760 \cdot cm^2$

Load:

$Q_d := 163 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.457 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.179 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$

$E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.623$ SS-EN 1993-1-3 5.2(5.2)

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The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.792 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

Flänsarnas bidrag enligt EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.867$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.867$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.695$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.797$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.905$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.835$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.742$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.952$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.009$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.684$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.835$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.835$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 3.186 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$$M_{Ed} < M_{f.Rd} = 0$$

$$V_{Ed} < V_{bw.RD} = 1$$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$$

From EWM calculations Ramboll

$$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$$

$$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$$

$$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.465 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.995$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

From EWM calculations Ramboll

$$\eta_3 := \frac{V_{Ed}}{V_{bw.RD}} = 0.684$$

EC1993-1-5 Ch. 7.1(5)

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 0$$

$$\eta_1 \ge \frac{M_{f.Rd}}{M_{pl.Rd}} = 1$$

$$\eta_1 + \left(1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 1.047$$

I-beam - EWM 500mm Shear capacity check

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\ m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 500mm$ $b_{2.st} := 500mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 760 \cdot cm^2$

Load:

$Q_d := 238 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 2.128 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 3.182 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$ $\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 2.37$ SS-EN 1993-1-3 5.2(5.2)

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The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 9.917 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 1.266$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 1.266$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 1.014$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 1.163$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 1.322$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.22$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 1.084$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 1.39$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 1.473$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.999$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.22$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 1.22$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 3.186 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.465 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 1.452$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.999$ EC1993-1-5 Ch. 7.1(5)

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 \le 1 = 0$ $\quad \eta_1 \ge \dfrac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot (2 \cdot \eta_3 - 1)^2 = 1.838$

I-beam - EWM 500mm Interaction

The I-beam is calculated with EWM at Ramboll. To be able to compare the RSM and the EWM some calculations are performed as a complement to the EWM calculations.

Geometry

Flange:

$t_f := 25mm$

$b_f := 560mm$ $b_{fu} := 560mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

$l_{beam} := 3 \cdot s_{ts} = 26.742\,m$

Measurements web plate between stiffeners:

$d_1 := 0.5m$ $d_2 := 0.5m$ $d_3 := 1m$

Horizontal stiffeners:

$b_{1.st} := 500mm$ $b_{2.st} := 500mm$ $t_{st} := 12mm$

$t_w := 12mm$ $h_w := 2m$

Total area:

$A_{tot} := t_w \cdot h_w + b_f \cdot t_f \cdot 2 + 2 \cdot t_{st} \cdot (b_{1.st} + b_{2.st}) = 760 \cdot cm^2$

Load:

$Q_d := 157 \frac{kN}{m}$

$M_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})^2}{8} = 1.403 \times 10^4 \cdot kN \cdot m$

$V_{Ed} := \frac{Q_d \cdot (3 \cdot s_{ts})}{2} = 2.099 \times 10^3 \cdot kN$

Material

$\gamma_M := 1$ $E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Capacities:

$M_{f.Rd} := 8.978 \cdot 10^3 kN \cdot m$ From EWM calculations

$V_{bw.Rd} := 2.513 \cdot 10^3 kN$ From EWM calculations

$\frac{M_{Ed}}{M_{f.Rd}} = 1.563$ SS-EN 1993-1-3 5.2(5.2)

p.86 BK

The contribution from the flanges:

$\frac{V_{Ed}}{l_{beam} \cdot t_w} = 6.542 \cdot MPa$

$V_{bf.Rd.ored} := 95.143kN$

Flänsarnas bidrag enligt EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Design resistance for shear:

$\eta := 1$

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_w) \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_M} = 1$ EC1993-1-5 Ch.5.2(1)

Shear capacity stiffened web

$V_{Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.513 \times 10^3 \cdot kN$

$\eta_3 := \frac{V_{Ed}}{V_{Rd}} = 0.835$ EN-1993-1-5 equ (5.8)

200mm stiffener

$V_{bw.Rd.200} := 2.513 \times 10^3\ kN$ (From Ramboll EWM calculations)

$\frac{V_{Ed}}{V_{bw.Rd}} = 0.835$

400 mm stiffener

$V_{bw.Rd.400} := 3.138 \cdot 10^3 kN$

$V_{Rd.400} := V_{bw.Rd.400} + V_{bf.Rd} = 3.138 \times 10^6 N$

$\frac{V_{Ed}}{V_{Rd.400}} = 0.669$

300 mm stiffener

$V_{bw.Rd.300} := 2.736 \cdot 10^3 kN$

$V_{Rd.300} := V_{bw.Rd.300} + V_{bf.Rd}$

$\frac{V_{Ed}}{V_{Rd.300}} = 0.767$

150 mm stiffener

$V_{bw.Rd.150} := 2.407 \cdot 10^3 kN$

$V_{Rd.150} := V_{bw.Rd.150} + V_{bf.Rd} = 2.407 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.150}} = 0.872$

250 mm stiffener

$V_{bw.Rd.250} := 2.609 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.805$

350 mm stiffener

$V_{bw.Rd.350} := 2.937 \cdot 10^3 kN$

$V_{Rd.350} := V_{bw.Rd.350} + V_{bf.Rd} = 2.937 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.350}} = 0.715$

100 mm stiffener

$V_{bw.Rd.100} := 2.289 \cdot 10^3 kN$

$V_{Rd.100} := V_{bw.Rd.100} + V_{bf.Rd} = 2.289 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.100}} = 0.917$

50 mm stiffener

$V_{bw.Rd.50} := 2.16 \cdot 10^3 kN$

$V_{Rd.50} := V_{bw.Rd.50} + V_{bf.Rd} = 2.16 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.50}} = 0.972$

500 mm stiffener

$V_{bw.Rd.500} := 3.186 \cdot 10^3 kN$

$V_{Rd.500} := V_{bw.Rd.500} + V_{bf.Rd} = 3.186 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.500}} = 0.659$

550 mm stiffener

$V_{bw.Rd.550} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.805$

600 mm stiffener

$V_{bw.Rd.600} := 3.186 \cdot 10^3 kN$

$V_{Rd.250} := V_{bw.Rd.250} + V_{bf.Rd} = 2.609 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{V_{Rd.250}} = 0.805$

V.bw.Rd, EWM with different stiffener widths

$$V_{bw.RD} := \begin{cases} V_{bw.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ V_{bw.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ V_{bw.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ V_{bw.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ V_{bw.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ V_{bw.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ V_{bw.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ V_{bw.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ V_{bw.Rd.500} & \text{otherwise} \end{cases} = 3.186 \times 10^{3} \cdot \text{kN} \cdot \text{m}$$

No interaction is needed because the load can be carried with the flanges contribution and the following conditions are met.

$M_{Ed} < M_{f.Rd} = 0$

$V_{Ed} < V_{bw.RD} = 1$

No interaction is needed if the load can be carried with the flanges contribution and the following conditions are met.

$$Check := \begin{cases} \text{"No interaction"} & \text{if } M_{Ed} < M_{f.Rd} \wedge V_{Ed} < V_{bw.RD} \\ \text{"No interaction"} & \text{if } V_{Ed} < 0.5 \cdot V_{bw.RD} \\ \text{"Interaction needed"} & \text{otherwise} \end{cases} = \text{"Interaction needed"}$$

SSEN1993-1-5 Ch.7.1(1)
Bärande K. Ch.S5.4 S86.

$M_{Rd} := 1.419 \cdot 10^{4} \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

$M_{pl.Rd.50} := 1.364 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.100} := 1.389 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.150} := 1.407 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.200} := M_{Rd} = 1.419 \times 10^{4} \cdot \text{kN} \cdot \text{m}$

$M_{pl.Rd.250} := 1.429 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.300} := 1.430 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.350} := 1.448 \times 10^{4} \text{ kN} \cdot \text{m}$

$M_{pl.Rd.400} := 1.454 \cdot 10^{4} \text{kN} \cdot \text{m}$

$M_{pl.Rd.500} := 1.465 \times 10^{4} \text{ kN} \cdot \text{m}$

M.Rd, EWM with different stiffener widths

$$M_{pl.Rd} := \begin{cases} M_{pl.Rd.50} & \text{if } b_{1.st} = 50\text{mm} \\ M_{pl.Rd.100} & \text{if } b_{1.st} = 100\text{mm} \\ M_{pl.Rd.150} & \text{if } b_{1.st} = 150\text{mm} \\ M_{pl.Rd.200} & \text{if } b_{1.st} = 200\text{mm} \\ M_{pl.Rd.250} & \text{if } b_{1.st} = 250\text{mm} \\ M_{pl.Rd.300} & \text{if } b_{1.st} = 300\text{mm} \\ M_{pl.Rd.350} & \text{if } b_{1.st} = 350\text{mm} \\ M_{pl.Rd.400} & \text{if } b_{1.st} = 400\text{mm} \\ M_{pl.Rd.500} & \text{otherwise} \end{cases} = 1.465 \times 10^{4} \cdot \text{kN} \cdot \text{m}$$

$\eta_1 := \dfrac{M_{Ed}}{M_{pl.Rd}} = 0.958$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

$M_{f.Rd} = 8.978 \times 10^{3} \cdot \text{kN} \cdot \text{m}$ From EWM calculations Ramboll

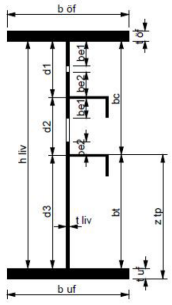
$\eta_3 := \dfrac{V_{Ed}}{V_{bw.RD}} = 0.659$ EC1993-1-5 Ch. 7.1(5)

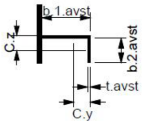
$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 \le 1 = 1$

$\eta_1 \ge \dfrac{M_{f.Rd}}{M_{pl.Rd}} = 1$

$\eta_1 + \left(1 - \dfrac{M_{f.Rd}}{M_{pl.Rd}}\right) \cdot \left(2 \cdot \eta_3 - 1\right)^2 = 0.997$

Calculation of a longitudinal stiffened I-beam with the RSM - 100mm horizontal stiffener





Material parameters:

$\gamma_M := 1$

$E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$f_{yk} := \begin{pmatrix} f_y(16mm) \\ f_y(40mm) \\ f_y(41mm) \end{pmatrix} = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Area

$A_{öf} := t_{öf} \cdot b_{öf}$ $A_{liv} := t_{web} \cdot h_{liv}$ $A_{uf} := t_{uf} \cdot b_{uf}$

$A_{avst} := t_{avst} \cdot (b_{1.avst} + b_{2.avst})$ $A_{tot} := A_{öf} + A_{liv} + A_{uf} + 2 \cdot A_{avst} = 5.536 \times 10^{4} \cdot mm^2$

Change of centre of gravity in y- and z-direction

$$C_y := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{b_{1.avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right)}{A_{avst}} = 12.786 \cdot mm$$

$$C_z := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(t_{avst} + \frac{b_{2.avst}}{2}\right)}{A_{avst}} = 38.786 \cdot mm$$

Geometry

$t_{öf} := 25mm$

$b_{öf} := 560mm$

$t_{web} := 12mm$

$h_{liv} := 2m$

$d_1 := 0.5m$

$d_2 := 0.5m$

$d_3 := 1m$

$t_{uf} := 25mm$

$b_{uf} := 560mm$

Horizontal stiffeners:

$b_{1.avst} := 50mm$

$b_{2.avst} := 90mm$

$t_{avst} := 12mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

Number of longitudinal stiffeners:

$n := 2$

Poissons ratio

$\nu := 0.3$

Distance from bottom fibre to centre of gravity

$$z_{tp} := \frac{\begin{matrix} A_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + A_{liv} \cdot \left(t_{uf} + \frac{h_{liv}}{2}\right) + A_{öf} \cdot \left(t_{uf} + h_{liv} + \frac{t_{öf}}{2}\right) \dots \\ + A_{avst} \cdot (t_{uf} + d_2 + d_3 - C_z) + A_{avst} \cdot (t_{uf} + d_3 - C_z) \end{matrix}}{A_{tot}} = 1.038\,m$$

Moment of inirtia, stiffeners $I_{z.avst}$

$$I_{z.avst} := \frac{b_{2.avst} \cdot t_{avst}^3}{12} + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2} - C_y\right)^2 \dots + \frac{t_{avst} \cdot b_{1.avst}^3}{12} + t_{avst} \cdot b_{1.avst} \cdot \left(\frac{b_{1.avst}}{2} - C_y\right)^2 = 2.772 \times 10^{-7}\,m^4$$

The stiffeners area and moment of inertia fig A.1

$b_1 := d_1$

$b_{3.c} := t_{uf} + d_3 + d_2 - z_{tp} = 0.487\,m$

Highest stress(under the flange) is set to the yield stress

$$\sigma_{max} := f_y = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Stress at a z-koord, bottom flange in tenstion

$$\sigma_{neg}(z) := \left(\frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}\right) \cdot -1$$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$

$$\psi_{neg}(z_1,z_2) := \frac{\sigma_{2.neg}(z_1,z_2)}{\sigma_{1.neg}(z_1,z_2)}$$

$$\psi_{liv.1} := \psi_{neg}(t_{uf}+d_2+d_3, t_{uf}+h_{liv}) = 0.453$$

$$\psi_{liv.2} := \psi_{neg}(t_{uf}+d_3, t_{uf}+d_2+d_3) = -0.037$$

$$\psi_1 := \psi_{liv.1} = 0.453$$

$$\psi_3 := \psi_{liv.2} = -0.037$$

$$b_{1.inf} := \frac{3-\psi_1}{5-\psi_1}\cdot b_1 = 280.065\cdot mm$$

$$b_{3.sup} := 0.4\cdot b_{3.c} = 192.91\cdot mm$$

$$A_{sl.1} := A_{avst} + t_{web}\cdot(b_{1.inf}+b_{3.sup}+t_{avst}) = 8.1\times10^{-3}\,m^2$$

$$y_{tp.sl.1} := \frac{t_{web}\cdot(b_{1.inf}+b_{3.sup}+t_{avst})\cdot\frac{t_{web}}{2} + A_{avst}\cdot(b_{1.avst}-C_y+t_{web})}{A_{sl.1}} = 27.63\cdot mm$$

Stiffeners moment of inertia out from the web plate

$$I_{z.sl.1} := I_{z.avst} + A_{avst}\cdot(b_{1.avst}-C_y+t_{web}-y_{tp.sl.1})^2 \ldots + \frac{(b_{1.inf}+b_{3.sup}+t_{avst})\cdot t_{web}^3}{12} \ldots + (b_{1.inf}+b_{3.sup}+t_{avst})\cdot t_{web}\cdot\left(\frac{t_{web}}{2}-y_{tp.sl.1}\right)^2 = 1.186\times10^{-5}\,m^4$$

Moment of inertia, Von mises stress and alpha critical

$q_d := 135\frac{kN}{m}$ $I_y := 0.0372m^4$ From EWM Ramboll

$\sigma_{v.unit} := \frac{2.4MPa}{\frac{kN}{m}}$ From FEM, close to middle of the span in top flange

$\alpha_{cr.UNIT} := 157.92\cdot\frac{kN}{m}$ Eigenvalue from FEM SOFiSTiK

$$\alpha_{cr_fem} := \frac{\alpha_{cr.UNIT}}{q_d} = 1.274$$

Load global

$$V_{Ed} := \frac{q_d\cdot(s_{ts}\cdot3)}{2} = 1.658\times10^3\cdot kN$$

$$M_{Ed} := \frac{q_d\cdot(s_{ts}\cdot3)^2}{8} = 1.108\times10^4\cdot kN\cdot m$$

$$V_{Ed.mid} := 0$$

Web

$$W_{y.web} := \frac{I_y}{h_{liv}+t_{uf}+t_{\ddot{o}f}-z_{tp}} = 0.037\cdot m^3$$

$$\sigma_{x.Ed.web} := \frac{M_{Ed}}{W_{y.web}} = 302.742\cdot MPa$$

$$\sigma_{z.Ed.web} := \frac{q_d}{t_{web}} = 10.333\cdot MPa$$

$$S := t_{\ddot{o}f}\cdot b_{\ddot{o}f}\cdot\left(t_{uf}+h_{liv}+\frac{t_{\ddot{o}f}}{2}-z_{tp}\right) + \frac{h_{liv}}{2}\cdot t_{web}\cdot\left(t_{uf}+\frac{h_{liv}}{2}+\frac{h_{liv}}{4}-z_{tp}\right) = 0.02\cdot m^3$$

$$\tau_{Ed.web} := \frac{V_{Ed}\cdot S}{I_y\cdot t_{web}} = 73.974\cdot MPa$$

$$\tau_{Ed.web.2mfromsupport} := \frac{(V_{Ed}-q_d\cdot2m)\cdot S}{I_y\cdot t_{web}} = 62.909\cdot MPa$$

Calculation of von mises stess analytical

$$\sigma_{v.analytical} := \sqrt{\sigma_{x.Ed.web}^2+\sigma_{z.Ed.web}^2-\sigma_{x.Ed.web}\cdot\sigma_{z.Ed.web}+3\cdot\tau_{Ed.web}^2} = 324.11\cdot MPa$$

Stiffeners

Stiffener over

$$W_{y.st.o} := \frac{I_y}{h_{liv}+t_{uf}-z_{tp}-d_1} = 0.076\cdot m^3$$

$$\sigma_{x.Ed.st.o} := \frac{M_{Ed}}{W_{y.st.o}} = 145.715\cdot MPa$$

Stiffener under

$$W_{y.st.u} := \frac{I_y}{h_{liv}+t_{uf}-z_{tp}-d_1-d_2} = -2.891\cdot m^3$$

$$\sigma_{x.Ed.st.u} := \frac{M_{Ed}}{W_{y.st.u}} = -3.834\cdot MPa$$

Flange

$$W_{y.fl} := \frac{I_y}{\frac{t_{\ddot{o}f}}{2}+t_{uf}+h_{liv}-z_{tp}} = 0.037\cdot m^3$$

$$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{W_{y.fl}} = 299.003\cdot MPa$$

$$S_{tp} := \frac{t_{\ddot{o}f}}{2}\cdot b_{\ddot{o}f}\cdot\left(\frac{t_{\ddot{o}f}}{2}+t_{uf}+h_{liv}-z_{tp}\right) = 6.998\times10^{-3}\cdot m^3$$

$$\tau_{Ed.tp.mid} := \frac{V_{Ed.mid}\cdot S_{tp}}{I_y\cdot b_{\ddot{o}f}} = 0\cdot MPa$$

$$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_y\cdot b_{\ddot{o}f}} = 0.559\cdot MPa$$

Calculation of load amplifiers, $\alpha_{ult,k}$, with von mises stress taken from sofistik.

-Formula for von mises in the denominator in α_{ultk}

$$\alpha_{ult.k_mises} := \frac{f_{y_2}}{\sigma_V} = 1.2$$

σ_V from sofistik EC3-1-5 Ch.10(4) eq.10.3

Global slenderness $\lambda_{p,mises}$

$$\lambda_p := \sqrt{\frac{\alpha_{ult.k_mises}}{\alpha_{cr_fem}}} = 0.971$$

Flange

Flange, calculation of $\sigma_{cr,p}$:

$\psi_{flange} := 1$ $k_{\sigma.flange} := 0.43$ Cross sectional parts in compression with free edge SS-EN 1993-1-5 table 4.2

$$\sigma_{E.flange} := \frac{\pi^2\cdot E\cdot t_{uf}^2}{12\cdot(1-\nu^2)\cdot\left(\frac{b_{uf}-t_{web}}{2}\right)^2} = 1.58\times10^3\cdot MPa$$

$$\sigma_{cr.p.fl} := k_{\sigma.flange}\cdot\sigma_{E.flange} = 679.428\cdot MPa$$

Flange; calculation of elastic critical column buckling stress $\sigma_{cr,c}$:

$$\sigma_{cr.c.fl} := \frac{\pi^2\cdot E\cdot t_{\ddot{o}f}^2}{12\cdot(1-\nu^2)\cdot s_{ts}^2} = 1.493\cdot MPa$$

Unstiffened plate

$k_{\sigma.st} := 0.43$

$\sigma_{E.st.1} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{1.avst})^2} = 2.733 \times 10^3 \cdot MPa$ Part 1 stiffener

$\sigma_{E.st.2} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{2.avst})^2} = 3.374 \times 10^3 \cdot MPa$ Part 2 stiffener

$\sigma_{cr.p.st.1} := k_{\sigma.st} \cdot \sigma_{E.st.1} = 1.175 \times 10^3 \cdot MPa$

$\sigma_{cr.p.st.2} := k_{\sigma.st} \cdot \sigma_{E.st.2} = 1.451 \times 10^3 \cdot MPa$

Stiffener; calculation of elastic critical column buckling stress $\sigma_{cr.c}$:

$\sigma_{cr.c.st} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$ Unstiffened plate

Calculation of the reduction factor χ_w for shear stess:

$\eta := 1.2$ EC3-1-5 Ch.5.1

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{0.7 + \lambda_p} & \text{otherwise} \end{cases}$$ EC3-1-5 tab.5.1

$\chi_w = 0.955$

WEB

Web; calculation of χ_{cx} (that depends on λ_p)

$\alpha := 0.49$

$e_2 := y_{tp.sl.1} - \frac{t_{web}}{2} = 9.652 \cdot mm$

$e_1 := C_y - e_2 - \frac{t_{web}}{2} = -2.866 \cdot mm$

$\delta := \max(e_1, e_2)$ EC3-1-5 4.5.3(5)

$i := \sqrt{\frac{I_{z.sl.1}}{A_{sl.1}}} = 0.019\,m$ EC3-1-5 4.5.3(5)

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.535$ EC3-1-5 4.5.3(5), stiffened plates

$\Phi_c := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 1.177$

$\chi_{cx} := \min\left(\frac{1}{\Phi_c + \sqrt{\Phi_c^2 - \lambda_p^2}}, 1\right) = 0.542$ EC3-1-1 eq. 6.49

Web; Calculation reduction factors ρ_p

Internal

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$ SS-EN 1993-1-5(eq4.2) Internal cross sectional parts

$\psi_{liv} := \psi_{neg}(t_{uf}, t_{uf} + h_{liv}) = -1.026$ Stress distribution, $\psi_x = \sigma_2/\sigma_1$

$\rho_{px} := \rho_{4.2}(\lambda_p, \psi_{liv}) = 0.915$

Calculation of amplifers, $\sigma_{cr.p}$ and $\sigma_{cr.c}$, for column like buckling in x direction for the plate

$b_2 := d_2 \qquad b := b_1 + b_2$

$a_c := 4.33 \cdot \sqrt[4]{\frac{I_{z.sl.1} \cdot b_1^2 \cdot b_2^2}{t_{web}^3 \cdot b}} = 2.439\,m$

$$\sigma_{cr.sl.p} := \begin{cases} \left(\frac{1.05 \cdot E}{A_{sl.1}} \cdot \frac{\sqrt{I_{z.sl.1} \cdot t_{web}^3 \cdot b}}{b_1 \cdot b_2}\right) & \text{if } s_{ts} \ge a_c \\ \left[\frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} + \frac{E \cdot t_{web}^3 \cdot b \cdot s_{ts}^2}{4 \cdot \pi^2 \cdot (1 - \nu^2) \cdot A_{sl.1} \cdot b_1^2 \cdot b_2^2}\right] & \text{if } s_{ts} \le a_c \end{cases}$$ Eq. A.4

$b_c := t_{uf} + h_{liv} - z_{tp} = 0.987\,m$

$b_{sl.1} := b_c - d_1 = 0.487\,m$

$b_t := h_{liv} - b_c = 1.013\,m$

$\sigma_{cr.p} := \sigma_{cr.sl.p} \cdot \frac{b_c}{b_{sl.1}} = 521.135 \cdot MPa$ Stiffened plate with one longitudinal stiffener in the compression zone. EC3-1-5 Ch. A.2.2

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} = 9.654 \cdot MPa$ Stiffened plate. EC3-1-5 Ch. 4.5.3

$\sigma_{cr.c} := \sigma_{cr.sl} \cdot \frac{b_c}{b_{sl.1}} = 19.562 \cdot MPa$

Interaction plate like/column like

$\xi_x := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.web} := (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} = 0.915$

$$\rho_{cx.web} := \begin{cases} (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} \ge 1 \end{cases} = 1$$ According to EC1993-1-5 eq.10.1

Transverse stresses (z-direction), calculation of ρ_{pz}:

$\lambda_{p0} := 0.8 \qquad \alpha_p := 0.34$ Table B.1

$\varphi_p := 0.5 \cdot \left[1 + \alpha_p \cdot (\lambda_p - \lambda_{p0}) + \lambda_p\right] = 1.014$

$$\rho_{p.z} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.796$$ EC1993-1-5, eq. B.1

Elastic critical buckling stress and the buckling stress for a planet that is not stiffened, seen from the z-direction:

$\sigma_{cr.c.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1 - \nu^2) \cdot h_{liv}^2} = 6.833 \cdot MPa$

$\sigma_{E.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$

$k_{\sigma z} := 4$

$\sigma_{cr.p.z} := k_{\sigma z} \cdot \sigma_{E.z} = 1.376 \cdot MPa$

Column buckling, χ_{cz}:

$\alpha_{unstiff} := 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.z} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot(\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.948$ Use λ_p according to EC3-1-5 Chapter 10

$\chi_{cz} := \dfrac{1}{\Phi_{c.z} + \sqrt{\Phi_{c.z}^{\,2} - \lambda_p^{\,2}}} = 0.754$ EC3-1-1 eq. 6.49

Interaction plate like/column like:

$\xi_z := \max\left(\min\left(\dfrac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 1\right), 0\right) = 0$

$\rho_{cz.web} := (\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz} = 0.754$

$\rho_{cz.web} := \begin{cases} (\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz} & \text{if } \dfrac{\left[(\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) \ldots + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \dfrac{\left[(\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} \geq 1 \end{cases} = 0.754$ According to EC1993-1-5 eq.10.1

The verification formula, web, with σ_{xEd}, σ_{zEd} and τ_{Ed} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots \leq 1 = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots = 0.76$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

Verification with V-factor according to Zizza

$c_{web} := \min\left(1.25 - 0.25\cdot\psi_{liv}, 1.25\right) = 1.25$

$\rho_{web.x.V} := \min\left[c_{web}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.996$

$\rho_{c.web.x.V} := \left(\rho_{web.x.V} - \chi_{cx}\right)\cdot\xi_x\cdot(2 - \xi_x) + \chi_{cx} = 0.996$

$\xi_{z.z} := \max\left(\dfrac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 0\right) = 0$

$f := \lambda_p^{\,-0.5\cdot\ln\left(\xi_{z.z} + 1\right)^{0.9}} = 0$

$\rho_{c.web.z.V} := \chi_{cz} + (\rho_{p.z} - \chi_{cz})\cdot f = 0.686$

$V_{web} := \rho_{c.web.x.V}\cdot\rho_{c.web.z.V} = 0.683$

$e_3 := 1.25 + 0.75\cdot\chi_w^{\,2} = 1.798$

The verification formula, web, V factor according to Zizza.

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots} \leq 1 = 1$$
$$+ -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots} = 0.898$$
$$+ -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}$$

Stiffeners

Reduction factors, cross sectional parts with free edge

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$ SS-EN 1993-1-5 eq 4.3 Cross sectional parts with free edge

$\psi := 1$

$\rho_{px.st.1} := \rho_{4.3}(\lambda_p, \psi) = 0.831$

Column buckling, $\chi_{cx.fl}$:

$\alpha_{unstiff} = 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.x.st.1} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot(\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 1.052$ Use λ_p EC3-1-5 Ch. 10

$\chi_{cx.st.1} := \dfrac{1}{\Phi_{c.x.st.1} + \sqrt{\Phi_{c.x.st.1}^{\,2} - \lambda_p^{\,2}}} = 0.686$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$\xi_{x.st.1} := \max\left(\min\left(\dfrac{\sigma_{cr.p.st.1}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.st.1} := \left(\rho_{px.st.1} - \chi_{cx.st.1}\right)\cdot\xi_{x.st.1}\cdot\left(2 - \xi_{x.st.1}\right) + \chi_{cx.st.1} = 0.831$ EC1993-1-5 eq.4.13

$\xi_{x.st.2} := \max\left(\min\left(\dfrac{\sigma_{cr.p.st.2}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$

The verification formula, stiffeners, with σ_{xEd} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \leq 1 = 1 \qquad \left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 = 0.213$$

Flange

Reduction factor, cross sectional parts with free edge

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$ SS-en 1993-1-5(eq4.3) cross sectional parts with free edge

$\psi := 1$

$\rho_{px.fl} := \rho_{4.3}(\lambda_p, \psi) = 0.831$

$$\Phi_{c.x.fl} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.948$$

Use λ_p, see EC3-1-5 Ch. 10

$$\chi_{cx.fl} := \frac{1}{\Phi_{c.x.fl} + \sqrt{\Phi_{c.x.fl}^{\,2} - \lambda_p^{\,2}}} = 0.754$$

EC3-1-1 eq. 6.49

Interaktion plate like/column like

$$\xi_{x.fl} := \max\left(\min\left(\frac{\sigma_{cr.p.fl}}{\sigma_{cr.c.fl}} - 1, 1\right), 0\right) = 1$$

Platelike behaviour

$$\rho_{cx.fl} := \left(\rho_{px.fl} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_{x.fl}\right) + \chi_{cx.fl} = 0.902$$

EC1993-1-5 eq.4.13

$$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^{\,2}, 2\right) = 2$$

The verification formula, flange:

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 \le 1 = 1$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 = 0.988$$

The verification formula, flange, V factor according to Zizza.

$\psi_{tp} := 1$ Uniform compression

$$c_{fl} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$$

$$\rho_{fl.x.V} := \min\left[c_{fl}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.86$$

$$c_{fl} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$$

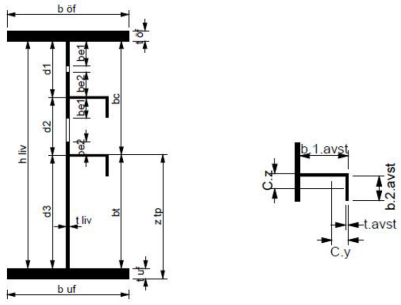
$$\rho_{fl.x.V} := \min\left[c_{fl}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.797$$

$$\rho_{c.fl.x.V} := \left(\rho_{fl.x.V} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_x\right) + \chi_{cx.fl} = 0.797$$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} \le 1 = 0$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.tp}}{\left(\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}\right)}\right]^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} = 1.042$$

Calculation of a longitudinal stiffened I-beam with the RSM - 100mm horizontal stiffener



Material parameters:

$\gamma_M := 1$

$E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$f_{\gamma M} := \begin{pmatrix} f_y(16mm) \\ f_y(40mm) \\ f_y(41mm) \end{pmatrix} = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Area

$A_{öf} := t_{öf} \cdot b_{öf}$ $A_{liv} := t_{web} \cdot h_{liv}$ $A_{uf} := t_{uf} \cdot b_{uf}$

$A_{avst} := t_{avst} \cdot (b_{1.avst} + b_{2.avst})$ $A_{tot} := A_{öf} + A_{liv} + A_{uf} + 2 \cdot A_{avst} = 5.656 \times 10^4 \cdot mm^2$

Change of centre of gravity in y och z-direction

$$C_y := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{b_{1.avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right)}{A_{avst}} = 29.158 \cdot mm$$

$$C_z := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(t_{avst} + \frac{b_{2.avst}}{2}\right)}{A_{avst}} = 30.158 \cdot mm$$

Geometry

$t_{öf} := 25mm$

$b_{öf} := 560mm$

$t_{web} := 12mm$

$h_{liv} := 2m$

$d_1 := 0.5m$

$d_2 := 0.5m$

$d_3 := 1m$

$t_{uf} := 25mm$

$b_{uf} := 560mm$

Horizontal stiffeners:

$b_{1.avst} := 100mm$

$b_{2.avst} := 90mm$

$t_{avst} := 12mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

Number of longitudinal stiffeners:

$n := 2$

Poissons ratio

$\nu := 0.3$

Distance from bottom fibre to centre of gravity

$$z_{tp} := \frac{A_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + A_{liv} \cdot \left(t_{uf} + \frac{h_{liv}}{2}\right) + A_{öf} \cdot \left(t_{uf} + h_{liv} + \frac{t_{öf}}{2}\right) \ldots}{A_{tot}} = 1.043\,m$$

$$\ldots + A_{avst} \cdot \left(t_{uf} + d_2 + d_3 - C_z\right) + A_{avst} \cdot \left(t_{uf} + d_3 - C_z\right)$$

Moment of inirtia, stiffeners $I_{z.avst}$

$$I_{z.avst} := \frac{b_{2.avst} \cdot t_{avst}^3}{12} + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2} - C_y\right)^2 + \frac{t_{avst} \cdot b_{1.avst}^3}{12} \ldots = 2.113 \times 10^{-6}\,m^4$$

$$\ldots + t_{avst} \cdot b_{1.avst} \cdot \left(\frac{b_{1.avst}}{2} - C_y\right)^2$$

The stiffeners area and moment of inertia fig A.1

$b_1 := d_1$

$b_{3.c} := t_{uf} + d_3 + d_2 - z_{tp} = 0.482\,m$

Highest stress(under the flange) is set to the yield stress

$$\sigma_{max} := f_y = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Stress at a z-koord, bottom flange in tenstion

$$\sigma_{neg}(z) := \left(\frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}\right) \cdot -1$$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$$

$$\psi_{liv.1} := \psi_{neg}(t_{uf} + d_2 + d_3, t_{uf} + h_{liv}) = 0.453$$

$$\psi_{liv.2} := \psi_{neg}(t_{uf} + d_3, t_{uf} + d_2 + d_3) = -0.037$$

$$\psi_1 := \psi_{liv.1} = 0.453$$

$$\psi_3 := \psi_{liv.2} = -0.037$$

$$b_{1.inf} := \frac{3 - \psi_1}{5 - \psi_1} \cdot b_1 = 280.065 \cdot mm$$

$$b_{3.sup} := 0.4 \cdot b_{3.c} = 192.91 \cdot mm$$

$$A_{sl.1} := A_{avst} + t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) = 8.1 \times 10^{-3}\, m^2$$

$$y_{tp.sl.1} := \frac{t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot \frac{t_{web}}{2} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web})}{A_{sl.1}} = 27.63 \cdot mm$$

Stiffeners moment of inertia out from the web plate

$$I_{z.sl.1} := I_{z.avst} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web} - y_{tp.sl.1})^2 \ldots + \frac{(b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web}^3}{12} \ldots + (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web} \cdot \left(\frac{t_{web}}{2} - y_{tp.sl.1}\right)^2 = 1.186 \times 10^{-5}\, m^4$$

Moment of inertia, Von mises stress and alpha critical

$q_d := 135 \frac{kN}{m}$ $I_y := 0.0372 m^4$ From EWM Ramboll

$$\sigma_{v.unit} := \frac{2.4 MPa}{\frac{kN}{m}}$$

From FEM, close to middle of the span in top flange

$$\sigma_V := \sigma_{v.unit} \cdot q_d = 324 \cdot MPa$$

$$\alpha_{cr.UNIT} := 198.76 \cdot \frac{kN}{m}$$

eigenvalue from FEM SOFiSTiK

$$\alpha_{cr_fem} := \frac{\alpha_{cr.UNIT}}{q_d} = 1.472$$

Load global

$$V_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)}{2} = 1.805 \times 10^3 \cdot kN$$

$$M_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)^2}{8} = 1.207 \times 10^4 \cdot kN \cdot m$$

$$V_{Ed.mid} := 0$$

Web

$$W_{y.web} := \frac{I_y}{h_{liv} + t_{uf} + t_{\ddot{o}f} - z_{tp}} = 0.037 \cdot m^3$$

$$\sigma_{x.Ed.web} := \frac{M_{Ed}}{W_{y.web}} = 326.766 \cdot MPa$$

$$\sigma_{z.Ed.web} := \frac{q_d}{t_{web}} = 11.25 \cdot MPa$$

$$S := t_{\ddot{o}f} \cdot b_{\ddot{o}f} \cdot \left(t_{uf} + h_{liv} + \frac{t_{\ddot{o}f}}{2} - z_{tp}\right) + \frac{h_{liv}}{2} \cdot t_{web} \cdot \left(t_{uf} + \frac{h_{liv}}{2} + \frac{h_{liv}}{4} - z_{tp}\right) = 0.02 \cdot m^3$$

$$\tau_{Ed.web} := \frac{V_{Ed} \cdot S}{I_y \cdot t_{web}} = 79.717 \cdot MPa$$

$$\tau_{Ed.web.2mfromsupport} := \frac{(V_{Ed} - q_d \cdot 2m) \cdot S}{I_y \cdot t_{web}} = 67.793 \cdot MPa$$

Calculation of .von mises stess analyttical

$$\sigma_{v.analytical} := \sqrt{\sigma_{x.Ed.web}^2 + \sigma_{z.Ed.web}^2 - \sigma_{x.Ed.web} \cdot \sigma_{z.Ed.web} + 3 \cdot \tau_{Ed.web}^2} = 349.701 \cdot MPa$$

Stiffeners

Stiffener over

$$W_{y.st.o} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1} = 0.077 \cdot m^3$$

$$\sigma_{x.Ed.st.o} := \frac{M_{Ed}}{W_{y.st.o}} = 156.453 \cdot MPa$$

Stiffener under

$$W_{y.st.u} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1 - d_2} = -2.099 \cdot m^3$$

$$\sigma_{x.Ed.st.u} := \frac{M_{Ed}}{W_{y.st.u}} = -5.75 \cdot MPa$$

Flange

$$W_{y.fl} := \frac{I_y}{\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}} = 0.037 \cdot m^3$$

$$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{W_{y.fl}} = 322.711 \cdot MPa$$

$$S_{tp} := \frac{t_{\ddot{o}f}}{2} \cdot b_{\ddot{o}f} \cdot \left(\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}\right) = 6.963 \times 10^{-3} \cdot m^3$$

$$\tau_{Ed.tp.mid} := \frac{V_{Ed.mid} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0 \cdot MPa$$

$$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0.603 \cdot MPa$$

Calculation of load amplifiers, $\alpha_{ult,k}$, with von mises stress taken from sofistik.

-Formula for von mises in the denominator in $\alpha_{ult.k}$

$$\alpha_{ult.k_mises} := \frac{f_{y2}}{\sigma_V} = 1.111$$

σ_V from sofistik EC3-1-5 Ch.10(4) eq.10.3

Global slenderness $\lambda_{p,mises}$

$$\lambda_p := \sqrt{\frac{\alpha_{ult.k_mises}}{\alpha_{cr_fem}}} = 0.869$$

Flange

Flange, calculation of $\sigma_{cr,p}$:

$\psi_{flange} := 1$ $k_{\sigma.flange} := 0.43$

Cross sectional parts in compression with free edge SS-EN 1993-1-5 table 4.2

$$\sigma_{E.flange} := \frac{\pi^2 \cdot E \cdot t_{uf}^2}{12 \cdot (1 - \nu^2) \cdot \left(\frac{b_{uf} - t_{web}}{2}\right)^2} = 1.58 \times 10^3 \cdot MPa$$

$$\sigma_{cr.p.fl} := k_{\sigma.flange} \cdot \sigma_{E.flange} = 679.428 \cdot MPa$$

Flange; calculation of elastic critical column buckling stress $\sigma_{cr,c}$:

$$\sigma_{cr.c.fl} := \frac{\pi^2 \cdot E \cdot t_{\ddot{o}f}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 1.493 \cdot MPa$$

Unstiffened plate

Stiffener; calculation of $\sigma_{cr,p}$:

$$\psi_{st} := 1$$

$k_{\sigma.st} := 0.43$

$\sigma_{E.st.1} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{1.avst})^2} = 2.733 \times 10^3 \cdot MPa$ Part 1 stiffener

$\sigma_{E.st.2} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{2.avst})^2} = 3.374 \times 10^3 \cdot MPa$ Part 2 stiffener

$\sigma_{cr.p.st.1} := k_{\sigma.st} \cdot \sigma_{E.st.1} = 1.175 \times 10^3 \cdot MPa$

$\sigma_{cr.p.st.2} := k_{\sigma.st} \cdot \sigma_{E.st.2} = 1.451 \times 10^3 \cdot MPa$

Stiffener; calculation of elastic critical column buckling stress $\sigma_{cr.c}$:

$\sigma_{cr.c.st} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$ Unstiffened plate

Calculation of the reduction factor χ_w for shear stess:

$\eta := 1.2$ EC3-1-5 Ch.5.1

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{0.7 + \lambda_p} & \text{otherwise} \end{cases}$$ EC3-1-5 tab.5.1

$\chi_w = 0.955$

WEB

Web; calculation of χ_{cx} (that depends on λ_p)

$\alpha := 0.49$

$e_2 := y_{tp.sl.1} - \frac{t_{web}}{2} = 21.63 \cdot mm$

$e_1 := C_y - e_2 - \frac{t_{web}}{2} = 1.527 \cdot mm$

$e := \max(e_1, e_2)$ EC3-1-5 4.5.3(5)

$i := \sqrt{\frac{I_{z.sl.1}}{A_{sl.1}}} = 0.038\,m$ EC3-1-5 4.5.3(5)

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.541$ EC3-1-5 4.5.3(5), för avstyvade plåtar

$\Phi_c := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 1.058$

$\chi_{cx} := \min\left(\frac{1}{\Phi_c + \sqrt{\Phi_c^2 - \lambda_p^2}}, 1\right) = 0.602$ EC3-1-1 eq. 6.49

Web; Calculation reduction factors ρ_p

Internal

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$ SS-en 1993-1-5(eq4.2) Internal cross sectional parts

$\psi_{liv} := \psi_{neg}(t_{uf}, t_{uf} + h_{liv}) = -1.036$ Stress distribution, $\psi_x = \sigma_2/\sigma_1$

$\rho_{px} := \rho_{4.2}(\lambda_p, \psi_{liv}) = 1$

Calculation of amplifers, $\sigma_{cr.p}$ and $\sigma_{cr.c}$, for column like buckling in x direction for the plate

$b_2 := d_2 \qquad b := b_1 + b_2$

$a_c := 4.33 \cdot \sqrt[4]{\frac{I_{z.sl.1} \cdot b_1^2 \cdot b_2^2}{t_{web}^3 \cdot b}} = 3.504\,m$

$$\sigma_{cr.sl.p} := \begin{cases} \left(\frac{1.05 \cdot E \cdot \sqrt{I_{z.sl.1} \cdot t_{web}^3 \cdot b}}{A_{sl.1} \cdot b_1 \cdot b_2}\right) & \text{if } s_{ts} \ge a_c \\ \left[\frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} + \frac{E \cdot t_{web}^3 \cdot b \cdot s_{ts}^2}{4 \cdot \pi^2 \cdot (1 - \nu^2) \cdot A_{sl.1} \cdot b_1^2 \cdot b_2^2}\right] & \text{if } s_{ts} \le a_c \end{cases}$$ Eq. A.4

$b_c := t_{uf} + h_{liv} - z_{tp} = 0.982\,m$

$b_{sl.1} := b_c - d_1 = 0.482\,m$

$b_t := h_{liv} - b_c = 1.018\,m$

$\sigma_{cr.p} := \sigma_{cr.sl.p} \cdot \frac{b_c}{b_{sl.1}} = 1.004 \times 10^3 \cdot MPa$ Stiffened plate with one longitudinal stiffener in the compression zone. EC3-1-5 Ch. A.2.2

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} = 38.182 \cdot MPa$ Stiffened plate. EC3-1-5 Ch. 4.5.3

$\sigma_{cr.c} := \sigma_{cr.sl} \cdot \frac{b_c}{b_{sl.1}} = 77.767 \cdot MPa$

Interaction plate like/column like

$\xi_x := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.web} := (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} = 1$

$$\rho_{cx.web} := \begin{cases} (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} \ge 1 \end{cases} = 1$$ According to EC1993-1-5 eq.10.1

Transverse stresses (z-direction), calculation of ρ_{pz}:

$\lambda_{p0} := 0.8 \qquad \alpha_p := 0.34$ Table B.1

$\varphi_p := 0.5 \cdot \left[1 + \alpha_p \cdot (\lambda_p - \lambda_{p0}) + \lambda_p\right] = 0.946$

$$\rho_{p.z} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.902$$ EC1993-1-5, eq. B.1

Elastic critical buckling stress and the buckling stress for a planet that is not stiffened, seen from the z-direction:

$\sigma_{cr.c.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1 - \nu^2) \cdot h_{liv}^2} = 6.833 \cdot MPa$

$\sigma_{E.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$

$k_{\sigma z} := 4$

$\sigma_{cr.p.z} := k_{\sigma z} \cdot \sigma_{E.z} = 1.376 \cdot MPa$

Column buckling, χ_{cz}:

$\alpha_{unstiff} := 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.z} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.948$ Use λ_p according to EC3-1-5 Chapter 10

$$\chi_{cz} := \frac{1}{\Phi_{c.z} + \sqrt{\Phi_{c.z}^{\,2} - \lambda_p^{\,2}}} = 0.754$$ EC3-1-1 eq. 6.49

Interaction plate like/column like:

$$\xi_z := \max\left(\min\left(\frac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 1\right), 0\right) = 0$$

$$\rho_{cz.web} := \left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz} = 0.754$$

$$\rho_{cz.web} := \begin{cases} \left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz} & \text{if } \dfrac{\left[\left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) \ldots\right] + \chi_{cz}}{\gamma_M}\cdot\alpha_{ult.k_mises} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} \geq 1 \end{cases} = 0.754$$ According to EC1993-1-5 eq.10.1

The verification formula, web, with σ_{xEd}, σ_{zEd} and τ_{Ed} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots \leq 1 = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots = 0.852$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

Verification with V-factor according to Zizza

$c_{web} := \min\left(1.25 - 0.25\cdot\psi_{liv}, 1.25\right) = 1.25$

$$\rho_{web.x.V} := \min\left[c_{web}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 1$$

$\rho_{c.web.x.V} := \left(\rho_{web.x.V} - \chi_{cx}\right)\cdot\xi_x\cdot\left(2 - \xi_x\right) + \chi_{cx} = 1$

$$\xi_{z.z} := \max\left(\frac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 0\right) = 0$$

$f := \lambda_p^{\,-0.5\cdot\ln\left(\xi_{z.z} + 1\right)^{0.9}} = 0$

$\rho_{c.web.z.V} := \chi_{cz} + \left(\rho_{p.z} - \chi_{cz}\right)\cdot f = 0.754$

$V_{web} := \rho_{c.web.x.V}\cdot\rho_{c.web.z.V} = 0.754$

$e_3 := 1.25 + 0.75\cdot\chi_w^{\,2} = 1.935$

The verification formula, web, V factor according to Zizza.

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} \leq 1 = 1$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} = 0.933$$

Stiffeners

Reduction factors, cross sectional parts with free edge

$$\rho_{4.3}\left(\lambda_p, \psi\right) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$$ SS-EN 1993-1-5 eq 4.3 Cross sectional parts with free edge

$\psi := 1$

$\rho_{px.st.1} := \rho_{4.3}\left(\lambda_p, \psi\right) = 0.902$

Column buckling, $\chi_{cx.fl}$:

$\alpha_{unstiff} = 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.x.st.1} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.948$ Use λ_p EC3-1-5 Ch. 10

$$\chi_{cx.st.1} := \frac{1}{\Phi_{c.x.st.1} + \sqrt{\Phi_{c.x.st.1}^{\,2} - \lambda_p^{\,2}}} = 0.754$$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$$\xi_{x.st.1} := \max\left(\min\left(\frac{\sigma_{cr.p.st.1}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$$

$\rho_{cx.st.1} := \left(\rho_{px.st.1} - \chi_{cx.st.1}\right)\cdot\xi_{x.st.1}\cdot\left(2 - \xi_{x.st.1}\right) + \chi_{cx.st.1} = 0.902$ EC1993-1-5 eq.4.13

$$\xi_{x.st.2} := \max\left(\min\left(\frac{\sigma_{cr.p.st.2}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$$

The verification formula, stiffeners, with σ_{xEd} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \leq 1 = 1 \qquad \left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 = 0.208$$

Flange

Reduction factor, cross sectional parts with free edge

$$\rho_{4.3}\left(\lambda_p, \psi\right) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$$ SS-en 1993-1-5(eq4.3) cross sectional parts with free edge

$\psi := 1$

$\rho_{px.fl} := \rho_{4.3}\left(\lambda_p, \psi\right) = 0.902$

Column buckling, $\chi_{cx.fl}$:

$\alpha_{unstiff} = 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$$\Phi_{c.x.fl} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.948$$

Use λ_p, see EC3-1-5 Ch. 10

$$\chi_{cx.fl} := \frac{1}{\Phi_{c.x.fl} + \sqrt{\Phi_{c.x.fl}^{\,2} - \lambda_p^{\,2}}} = 0.754$$

EC3-1-1 eq. 6.49

Interaktion plate like/column like

$$\xi_{x.fl} := \max\left(\min\left(\frac{\sigma_{cr.p.fl}}{\sigma_{cr.c.fl}} - 1, 1\right), 0\right) = 1$$

Platelike behaviour

$$\rho_{cx.fl} := \left(\rho_{px.fl} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_{x.fl}\right) + \chi_{cx.fl} = 0.902$$

EC1993-1-5 eq.4.13

$$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^{\,2}, 2\right) = 2$$

The verification formula, flange:

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 \le 1 = 1$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 = 0.988$$

The verification formula, flange, V factor according to Zizza.

$\psi_{tp} := 1$ Uniform compression

$$c_{fl} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$$

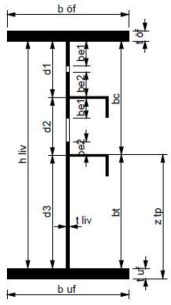
$$\rho_{fl.x.V} := \min\left[c_{fl}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.86$$

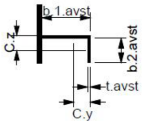
$$\rho_{c.fl.x.V} := \left(\rho_{fl.x.V} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_x\right) + \chi_{cx.fl} = 0.86$$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} \le 1 = 0$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.tp}}{\left(\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}\right)}\right]^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} = 1.043$$

Calculation of a longitudinal stiffened I-beam with the RSM - 150mm horizontal stiffener





Material parameters:

$\gamma_M := 1$

$E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$f_{yk} := \begin{pmatrix} f_y(16mm) \\ f_y(40mm) \\ f_y(41mm) \end{pmatrix} = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Area

$A_{öf} := t_{öf} \cdot b_{öf}$ $A_{liv} := t_{web} \cdot h_{liv}$ $A_{uf} := t_{uf} \cdot b_{uf}$

$A_{avst} := t_{avst} \cdot (b_{1.avst} + b_{2.avst})$ $A_{tot} := A_{öf} + A_{liv} + A_{uf} + 2 \cdot A_{avst} = 5.776 \times 10^4 \cdot mm^2$

Change of centre of gravity in y- and z-direction

$$C_y := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{b_{1.avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right)}{A_{avst}} = 49.125 \cdot mm$$

$$C_z := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(t_{avst} + \frac{b_{2.avst}}{2}\right)}{A_{avst}} = 25.125 \cdot mm$$

Geometry

$t_{öf} := 25mm$

$b_{öf} := 560mm$

$t_{web} := 12mm$

$h_{liv} := 2m$

$d_1 := 0.5m$

$d_2 := 0.5m$

$d_3 := 1m$

$t_{uf} := 25mm$

$b_{uf} := 560mm$

Horizontal stiffeners:

$b_{1.avst} := 150mm$

$b_{2.avst} := 90mm$

$t_{avst} := 12mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

Number of longitudinal stiffeners:

$n := 2$

Poissons ratio

$\nu := 0.3$

Distance from bottom fibre to centre of gravity

$$z_{tp} := \frac{A_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + A_{liv} \cdot \left(t_{uf} + \frac{h_{liv}}{2}\right) + A_{öf} \cdot \left(t_{uf} + h_{liv} + \frac{t_{öf}}{2}\right) \ldots + A_{avst} \cdot (t_{uf} + d_2 + d_3 - C_z) + A_{avst} \cdot (t_{uf} + d_3 - C_z)}{A_{tot}} = 1.047\,m$$

Moment of inirtia, stiffeners $I_{z.avst}$

$$I_{z.avst} := \frac{b_{2.avst} \cdot t_{avst}^3}{12} + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2} - C_y\right)^2 \ldots + \frac{t_{avst} \cdot b_{1.avst}^3}{12} + t_{avst} \cdot b_{1.avst} \cdot \left(\frac{b_{1.avst}}{2} - C_y\right)^2 = 6.602 \times 10^{-6}\,m^4$$

The stiffeners area and moment of inertia fig A.1

$b_1 := d_1$

$b_{3.c} := t_{uf} + d_3 + d_2 - z_{tp} = 0.478\,m$

Highest stress(under the flange) is set to the yield stress

$$\sigma_{max} := f_y = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Stress at a z-koord, bottom flange in tenstion

$$\sigma_{neg}(z) := \left(\frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}\right) \cdot -1$$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$$

$\psi_{liv.1} := \psi_{neg}(t_{uf} + d_2 + d_3, t_{uf} + h_{liv}) = 0.451$

$\psi_{liv.2} := \psi_{neg}(t_{uf} + d_3, t_{uf} + d_2 + d_3) = -0.047$

$\psi_1 := \psi_{liv.1} = 0.451$

$\psi_3 := \psi_{liv.2} = -0.047$

$b_{1.inf} := \frac{3 - \psi_1}{5 - \psi_1} \cdot b_1 = 280.174 \cdot mm$

$b_{3.sup} := 0.4 \cdot b_{3.c} = 191.03 \cdot mm$

$A_{sl.1} := A_{avst} + t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) = 8.678 \times 10^{-3} m^2$

$y_{tp.sl.1} := \frac{t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot \frac{t_{web}}{2} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web})}{A_{sl.1}} = 41.467 \cdot mm$

Stiffeners moment of inertia out from the web plate

$I_{z.sl.1} := I_{z.avst} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web} - y_{tp.sl.1})^2 \ldots + \frac{(b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web}^3}{12} \ldots + (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web} \cdot \left(\frac{t_{web}}{2} - y_{tp.sl.1}\right)^2 = 2.865 \times 10^{-5} m^4$

Moment of inertia, Von mises stress and alpha critical

$q_d := 146 \frac{kN}{m}$ $I_y := 0.0373 m^4$ From EWM Ramboll

$\sigma_{v.unit} := \frac{2.37 MPa}{\frac{kN}{m}}$ From FEM, close to middle of the span in top flange

$\sigma_V := \sigma_{v.unit} \cdot q_d = 346.02 \cdot MPa$

$\alpha_{cr.UNIT} := 244.13 \cdot \frac{kN}{m}$ Eigenvalue from FEM SOFiSTiK

$\alpha_{cr_fem} := \frac{\alpha_{cr.UNIT}}{q_d} = 1.672$

Load global

$V_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)}{2} = 1.952 \times 10^3 \cdot kN$

$M_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)^2}{8} = 1.305 \times 10^4 \cdot kN \cdot m$

$V_{Ed.mid} := 0$

Web

$W_{y.web} := \frac{I_y}{h_{liv} + t_{uf} + t_{\ddot{o}f} - z_{tp}} = 0.037 \cdot m^3$

$\sigma_{x.Ed.web} := \frac{M_{Ed}}{W_{y.web}} = 350.799 \cdot MPa$

$\sigma_{z.Ed.web} := \frac{q_d}{t_{web}} = 12.167 \cdot MPa$

$S := t_{\ddot{o}f} \cdot b_{\ddot{o}f} \cdot \left(t_{uf} + h_{liv} + \frac{t_{\ddot{o}f}}{2} - z_{tp}\right) + \frac{h_{liv}}{2} \cdot t_{web} \cdot \left(t_{uf} + \frac{h_{liv}}{2} + \frac{h_{liv}}{4} - z_{tp}\right) = 0.02 \cdot m^3$

$\tau_{Ed.web} := \frac{V_{Ed} \cdot S}{I_y \cdot t_{web}} = 85.448 \cdot MPa$

$\tau_{Ed.web.2mfromsupport} := \frac{(V_{Ed} - q_d \cdot 2m) \cdot S}{I_y \cdot t_{web}} = 72.667 \cdot MPa$

Calculation of .von mises stess analyttical

$\sigma_{v.analytical} := \sqrt{\sigma_{x.Ed.web}^2 + \sigma_{z.Ed.web}^2 - \sigma_{x.Ed.web} \cdot \sigma_{z.Ed.web} + 3 \cdot \tau_{Ed.web}^2} = 375.292 \cdot MPa$

Stiffeners

Stiffener over

$W_{y.st.o} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1} = 0.078 \cdot m^3$

$\sigma_{x.Ed.st.o} := \frac{M_{Ed}}{W_{y.st.o}} = 167.103 \cdot MPa$

Stiffener under

$W_{y.st.u} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1 - d_2} = -1.663 \cdot m^3$

$\sigma_{x.Ed.st.u} := \frac{M_{Ed}}{W_{y.st.u}} = -7.847 \cdot MPa$

Flange

$W_{y.fl} := \frac{I_y}{\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}} = 0.038 \cdot m^3$

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{W_{y.fl}} = 346.425 \cdot MPa$

$S_{tp} := \frac{t_{\ddot{o}f}}{2} \cdot b_{\ddot{o}f} \cdot \left(\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}\right) = 6.931 \times 10^{-3} \cdot m^3$

$\tau_{Ed.tp.mid} := \frac{V_{Ed.mid} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0 \cdot MPa$

$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0.648 \cdot MPa$

Calculation of load amplifiers, $\alpha_{ult,k}$, with von mises stress taken from sofistik.

-Formula for von mises in the denominator in α_{ultk}

$\alpha_{ult.k_mises} := \frac{f_{y2}}{\sigma_V} = 1.04$ σ_V from sofistik EC3-1-5 Ch.10(4) eq.10.3

Global slenderness $\lambda_{p,mises}$

$\lambda_p := \sqrt{\frac{\alpha_{ult.k_mises}}{\alpha_{cr_fem}}} = 0.789$

Flange

Flange, calculation of $\sigma_{cr,p}$:

$\psi_{flange} := 1$ $k_{\sigma.flange} := 0.43$ Cross sectional parts in compression with free edge SS-EN 1993-1-5 table 4.2

$\sigma_{E.flange} := \frac{\pi^2 \cdot E \cdot t_{uf}^2}{12 \cdot (1 - \nu^2) \cdot \left(\frac{b_{uf} - t_{web}}{2}\right)^2} = 1.58 \times 10^3 \cdot MPa$

$\sigma_{cr.p.fl} := k_{\sigma.flange} \cdot \sigma_{E.flange} = 679.428 \cdot MPa$

Flange; calculation of elastic critical column buckling stress $\sigma_{cr,c}$:

$\sigma_{cr.c.fl} := \frac{\pi^2 \cdot E \cdot t_{\ddot{o}f}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 1.493 \cdot MPa$ Unstiffened plate

Stiffener; calculation of $\sigma_{cr,p}$:

$\psi_{st} := 1$

$k_{\sigma.st} := 0.43$

$\sigma_{E.st.1} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{1.avst})^2} = 1.215 \times 10^3 \cdot MPa$ Part 1 stiffener

$$\sigma_{E.st.2} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1-\nu^2) \cdot (b_{2.avst})^2} = 3.374 \times 10^3 \cdot MPa$$ Part 2 stiffener

$$\sigma_{cr.p.st.1} := k_{\sigma.st} \cdot \sigma_{E.st.1} = 522.33 \cdot MPa$$

$$\sigma_{cr.p.st.2} := k_{\sigma.st} \cdot \sigma_{E.st.2} = 1.451 \times 10^3 \cdot MPa$$

Stiffener; calculation of elastic critical column buckling stress $\sigma_{cr.c}$:

$$\sigma_{cr.c.st} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1-\nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$$ Unstiffened plate

Calculation of the reduction factor χ_w for shear stess:

$\eta := 1.2$ EC3-1-5 Ch.5.1

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{0.7 + \lambda_p} & \text{otherwise} \end{cases}$$ EC3-1-5 tab.5.1

$\chi_w = 1.052$

WEB

Web; calculation of χ_{cx} (that depends on λ_p)

$\alpha := 0.49$

$$e_2 := y_{tp.sl.1} - \frac{t_{web}}{2} = 35.467 \cdot mm$$

$$e_1 := C_y - e_2 - \frac{t_{web}}{2} = 7.658 \cdot mm$$

$$\xi := \max(e_1, e_2)$$ EC3-1-5 4.5.3(5)

$$i := \sqrt{\frac{I_{z.sl.1}}{A_{sl.1}}} = 0.057\,m$$ EC3-1-5 4.5.3(5)

$$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.546$$ EC3-1-5 4.5.3(5), för stiffened plates

$$\Phi_c := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.972$$

$$\chi_{cx} := \min\left(\frac{1}{\Phi_c + \sqrt{\Phi_c^2 - \lambda_p^2}}, 1\right) = 0.65$$ EC3-1-1 eq. 6.49

Web; Calculation reduction factors ρ_p

Internal

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max[(3+\psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$ SS-en 1993-1-5(eq4.2) Internal cross sectional parts

$\psi_{liv} := \psi_{neg}(t_{uf}, t_{uf} + h_{liv}) = -1.046$ Stress distribution, $\psi_x = \sigma_2/\sigma_1$

$\rho_{px} := \rho_{4.2}(\lambda_p, \psi_{liv}) = 1$

Calculation of amplifers, $\sigma_{cr.p}$ and $\sigma_{cr.c}$, for column like buckling in x direction for the plate

$b_2 := d_2 \qquad b := b_1 + b_2$

$$a_c := 4.33 \cdot \sqrt[4]{\frac{I_{z.sl.1} \cdot b_1^2 \cdot b_2^2}{t_{web}^3 \cdot b}} = 4.369\,m$$

$$\sigma_{cr.sl.p} := \begin{cases} \left(\frac{1.05 \cdot E}{A_{sl.1}} \cdot \sqrt{\frac{I_{z.sl.1} \cdot t_{web}^3 \cdot b}{b_1 \cdot b_2}}\right) & \text{if } s_{ts} \ge a_c \\ \left[\frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} + \frac{E \cdot t_{web}^3 \cdot b \cdot s_{ts}^2}{4 \cdot \pi^2 \cdot (1-\nu^2) \cdot A_{sl.1} \cdot b_1^2 \cdot b_2^2}\right] & \text{if } s_{ts} \le a_c \end{cases}$$ Eq. A.4

$$b_c := t_{uf} + h_{liv} - z_{tp} = 0.978\,m$$

$$b_{sl.1} := b_c - d_1 = 0.478\,m$$

$$b_t := h_{liv} - b_c = 1.022\,m$$

$$\sigma_{cr.p} := \sigma_{cr.sl.p} \cdot \frac{b_c}{b_{sl.1}} = 1.464 \times 10^3 \cdot MPa$$ Stiffened plate with one longitudinal stiffener in the compression zone. EC3-1-5 Ch. A.2.2

$$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} = 86.112 \cdot MPa$$ Stiffened plate. EC3-1-5 Ch. 4.5.3

$$\sigma_{cr.c} := \sigma_{cr.sl} \cdot \frac{b_c}{b_{sl.1}} = 176.268 \cdot MPa$$

Interaction plate like/column like

$$\xi_x := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1, 1\right), 0\right) = 1$$

$$\rho_{cx.web} := (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} = 1$$

$$\rho_{cx.web} := \begin{cases} (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} \ge 1 \end{cases} = 1$$ According to EC1993-1-5 eq.10.1

Transverse stresses (z-direction), calculation of ρ_{pz}:

$\lambda_{p0} := 0.8 \qquad \alpha_p := 0.34$ Table B.1

$$\varphi_p := 0.5 \cdot \left[1 + \alpha_p \cdot (\lambda_p - \lambda_{p0}) + \lambda_p\right] = 0.892$$

$$\rho_{p.z} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.02$$ EC1993-1-5, eq. B.1

Elastic critical buckling stress and the buckling stress for a planet that is not stiffened, seen from the z-direction:

$$\sigma_{cr.c.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1-\nu^2) \cdot h_{liv}^2} = 6.833 \cdot MPa$$

$$\sigma_{E.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1-\nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$$

$k_{\sigma z} := 4$

$\sigma_{cr.p.z} := k_{\sigma z} \cdot \sigma_{E.z} = 1.376 \cdot MPa$

Column buckling, χ_{cz}:

$\alpha_{unstiff} := 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.z} := 0.5 \cdot \left[1 + \alpha_{unstiff} \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.873$ Use λ_p according to EC3-1-5 Chapter 10

$$\chi_{cz} := \frac{1}{\Phi_{c.z} + \sqrt{\Phi_{c.z}^2 - \lambda_p^2}} = 0.802$$ EC3-1-1 eq. 6.49

Interaction plate like/column like:

$$\xi_z := \max\left(\min\left(\frac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 1\right), 0\right) = 0$$

$$\rho_{cz.web} := \left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz} = 0.802$$

$$\rho_{cz.web} := \begin{cases} \left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz} & \text{if } \frac{\left[\left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) \ldots \right]\cdot\alpha_{ult.k_mises}}{\gamma_M} < 1 \quad = 0.80 \\ 1 & \text{if } \frac{\left[\left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} \geq 1 \end{cases}$$

According to EC1993-1-5 eq.10.1

The verification formula, web, with σ_{xEd}, σ_{zEd} and τ_{Ed} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots \leq 1 = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots = 0.954$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

Verification with V-factor according to Zizza

$$c_{web} := \min\left(1.25 - 0.25\cdot\psi_{liv}, 1.25\right) = 1.25$$

$$\rho_{web.x.V} := \min\left[c_{web}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$$

$$\rho_{c.web.x.V} := \left(\rho_{web.x.V} - \chi_{cx}\right)\cdot\xi_x\cdot\left(2 - \xi_x\right) + \chi_{cx} = 1$$

$$\xi_{z.z} := \max\left(\frac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 0\right) = 0$$

$$f := \lambda_p^{-0.5}\cdot\ln\left(\xi_{z.z} + 1\right)^{0.9} = 0$$

$$\rho_{c.web.z.V} := \chi_{cz} + \left(\rho_{p.z} - \chi_{cz}\right)\cdot f = 0.802$$

$$V_{web} := \rho_{c.web.x.V}\cdot\rho_{c.web.z.V} = 0.802$$

$$e_3 := 1.25 + 0.75\cdot\chi_w^2 = 2.08$$

The verification formula, web, V factor according to Zizza.

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots} \leq 1 = 1$$
$$\sqrt{\ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}}$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots} = 0.975$$
$$\sqrt{\ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}}$$

Stiffeners

Reduction factors, cross sectional parts with free edge

$$\rho_{4.3}\left(\lambda_p, \psi\right) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 eq 4.3
Cross sectional parts with free edge

$$\psi := 1$$

$$\rho_{px.st.1} := \rho_{4.3}\left(\lambda_p, \psi\right) = 0.966$$

Column buckling, $\chi_{cx.ft}$

$$\alpha_{unstiff} = 0.21$$

EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$$\Phi_{c.x.st.1} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.873$$

Use λ_p, EC3-1-5 Ch. 10

$$\chi_{cx.st.1} := \frac{1}{\Phi_{c.x.st.1} + \sqrt{\Phi_{c.x.st.1}^2 - \lambda_p^2}} = 0.802$$

EC3-1-1 eq. 6.49

Interaction plate like/column like

$$\xi_{x.st.1} := \max\left(\min\left(\frac{\sigma_{cr.p.st.1}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$$

$$\rho_{cx.st.1} := \left(\rho_{px.st.1} - \chi_{cx.st.1}\right)\cdot\xi_{x.st.1}\cdot\left(2 - \xi_{x.st.1}\right) + \chi_{cx.st.1} = 0.966$$

EC1993-1-5 eq.4.13

$$\xi_{x.st.2} := \max\left(\min\left(\frac{\sigma_{cr.p.st.2}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$$

The verification formula, stiffeners, with σ_{xEd} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \leq 1 = 1 \qquad \left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 = 0.207$$

Flange

Reduction factor, cross sectional parts with free edge

$$\rho_{4.3}\left(\lambda_p, \psi\right) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-en 1993-1-5(eq4.3) cross sectional parts with free edge

$$\psi := 1$$

$$\rho_{px.fl} := \rho_{4.3}\left(\lambda_p, \psi\right) = 0.966$$

Column buckling, $\chi_{cx.ft}$

$$\alpha_{unstiff} = 0.21$$

EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$$\Phi_{c.x.fl} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.873$$

Use λ_p, see EC3-1-5 Ch. 10

$$\chi_{cx.fl} := \frac{1}{\Phi_{c.x.fl} + \sqrt{\Phi_{c.x.fl}^2 - \lambda_p^2}} = 0.802$$

EC3-1-1 eq. 6.49

Interaction plate like/column like

$$\xi_{x.fl} := \max\left(\min\left(\frac{\sigma_{cr.p.fl}}{\sigma_{cr.c.fl}} - 1, 1\right), 0\right) = 1$$

Platelike behaviour

$\rho_{cx.fl} := \left(\rho_{px.fl} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_{x.fl}\right) + \chi_{cx.fl} = 0.966$ EC1993-1-5 eq.4.13

$\lambda_{aw} := \max\left(1.25 + 0.75\cdot\chi_w^{\,2}, 2\right) = 2.08$

The verification formula, flange:

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 \le 1 = 1$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 = 0.993$$

The verification formula, flange, V factor according to Zizza.

$\psi_{tp} := 1$ Uniform compression

$c_{fl} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

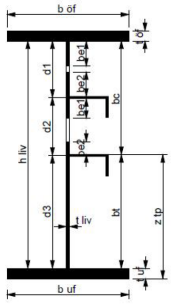
$\rho_{fl.x.V} := \min\left[c_{fl}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.914$

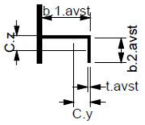
$\rho_{c.fl.x.V} := \left(\rho_{fl.x.V} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_x\right) + \chi_{cx.fl} = 0.914$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} \le 1 = 0$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.tp}}{\left(\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}\right)}\right]^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} = 1.053$$

Calculation of a longitudinal stiffened I-beam with the RSM - 200mm horizontal stiffener





Material parameters:

$\gamma_M := 1$

$E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$f_{y.M} := \begin{pmatrix} f_y(16mm) \\ f_y(40mm) \\ f_y(41mm) \end{pmatrix} = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Area

$A_{öf} := t_{öf} \cdot b_{öf}$ $A_{liv} := t_{web} \cdot h_{liv}$ $A_{uf} := t_{uf} \cdot b_{uf}$

$A_{avst} := t_{avst} \cdot (b_{1.avst} + b_{2.avst})$ $A_{tot} := A_{öf} + A_{liv} + A_{uf} + 2 \cdot A_{avst} = 5.896 \times 10^{4} \cdot mm^2$

Change of centre of gravity in y- and z-direction

$$C_y := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{b_{1.avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right)}{A_{avst}} = 70.828 \cdot mm$$

$$C_z := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(t_{avst} + \frac{b_{2.avst}}{2}\right)}{A_{avst}} = 21.828 \cdot mm$$

Geometry

$t_{öf} := 25mm$

$b_{öf} := 560mm$

$t_{web} := 12mm$

$h_{liv} := 2m$

$d_1 := 0.5m$

$d_2 := 0.5m$

$d_3 := 1m$

$t_{uf} := 25mm$

$b_{uf} := 560mm$

Horizontal stiffeners:

$b_{1.avst} := 200mm$

$b_{2.avst} := 90mm$

$t_{avst} := 12mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

Number of longitudinal stiffeners:

$n := 2$

Poissons ratio

$\nu := 0.3$

Distance from bottom fibre to centre of gravity

$$z_{tp} := \frac{A_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + A_{liv} \cdot \left(t_{uf} + \frac{h_{liv}}{2}\right) + A_{öf} \cdot \left(t_{uf} + h_{liv} + \frac{t_{öf}}{2}\right) \ldots + A_{avst} \cdot \left(t_{uf} + d_2 + d_3 - C_z\right) + A_{avst} \cdot \left(t_{uf} + d_3 - C_z\right)}{A_{tot}} = 1.052\,m$$

Moment of inirtia, stiffeners $I_{z.avst}$

$$I_{z.avst} := \frac{b_{2.avst} \cdot t_{avst}^3}{12} + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2} - C_y\right)^2 \ldots + \frac{t_{avst} \cdot b_{1.avst}^3}{12} + t_{avst} \cdot b_{1.avst} \cdot \left(\frac{b_{1.avst}}{2} - C_y\right)^2 = 1.459 \times 10^{-5}\,m^4$$

The stiffeners area and moment of inertia fig A.1

$b_1 := d_1$

$b_{3.c} := t_{uf} + d_3 + d_2 - z_{tp} = 0.473\,m$

Highest stress(under the flange) is set to the yield stress

$$\sigma_{max} := f_y = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Stress at a z-koord, bottom flange in tenstion

$$\sigma_{neg}(z) := \left(\frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}\right) \cdot -1$$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$$

$\psi_{liv.1} := \psi_{neg}(t_{uf} + d_2 + d_3, t_{uf} + h_{liv}) = 0.449$

$\psi_{liv.2} := \psi_{neg}(t_{uf} + d_3, t_{uf} + d_2 + d_3) = -0.057$

$\psi_1 := \psi_{liv.1} = 0.449$

$\psi_3 := \psi_{liv.2} = -0.057$

$b_{1.inf} := \frac{3 - \psi_1}{5 - \psi_1} \cdot b_1 = 280.28 \cdot mm$

$b_{3.sup} := 0.4 \cdot b_{3.c} = 189.226 \cdot mm$

$A_{sl.1} := A_{avst} + t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) = 9.258 \times 10^{-3} m^2$

$y_{tp.sl.1} := \frac{t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot \frac{t_{web}}{2} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web})}{A_{sl.1}} = 56.81 \cdot mm$

Stiffeners moment of inertia out from the web plate

$I_{z.sl.1} := I_{z.avst} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web} - y_{tp.sl.1})^2 \ldots + \frac{(b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web}^3}{12} \ldots + (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web} \cdot \left(\frac{t_{web}}{2} - y_{tp.sl.1}\right)^2 = 5.435 \times 10^{-5} m^4$

Moment of inertia, Von mises stress and alpha critical

$q_d := 153 \frac{kN}{m}$ $I_y := 0.0375 m^4$ From the EWM Ramboll calculations

$\sigma_{v.unit} := \frac{2.36 MPa}{\frac{kN}{m}}$ From FEM, close to middle of the span in top flange

$\sigma_V := \sigma_{v.unit} \cdot q_d = 361.08 \cdot MPa$

$\alpha_{cr.UNIT} := 304.46 \cdot \frac{kN}{m}$ Eigenvalue from FEM SOFiSTiK

$\alpha_{cr_fem} := \frac{\alpha_{cr.UNIT}}{q_d} = 1.99$

Load global

$V_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)}{2} = 2.046 \times 10^3 \cdot kN$

$M_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)^2}{8} = 1.368 \times 10^4 \cdot kN \cdot m$

$V_{Ed.mid} := 0$

Web

$W_{y.web} := \frac{I_y}{h_{liv} + t_{uf} + t_{\ddot{o}f} - z_{tp}} = 0.038 \cdot m^3$

$\sigma_{x.Ed.web} := \frac{M_{Ed}}{W_{y.web}} = 364.013 \cdot MPa$

$\sigma_{z.Ed.web} := \frac{q_d}{t_{web}} = 12.75 \cdot MPa$

$S := t_{\ddot{o}f} \cdot b_{\ddot{o}f} \cdot \left(t_{uf} + h_{liv} + \frac{t_{\ddot{o}f}}{2} - z_{tp}\right) + \frac{h_{liv}}{2} \cdot t_{web} \cdot \left(t_{uf} + \frac{h_{liv}}{2} + \frac{h_{liv}}{4} - z_{tp}\right) = 0.019 \cdot m^3$

$\tau_{Ed.web} := \frac{V_{Ed} \cdot S}{I_y \cdot t_{web}} = 88.535 \cdot MPa$

$\tau_{Ed.web.2mfromsupport} := \frac{(V_{Ed} - q_d \cdot 2m) \cdot S}{I_y \cdot t_{web}} = 75.292 \cdot MPa$

Calculation of .von mises stess analyttical

$\sigma_{v.analytical} := \sqrt{\sigma_{x.Ed.web}^2 + \sigma_{z.Ed.web}^2 - \sigma_{x.Ed.web} \cdot \sigma_{z.Ed.web} + 3 \cdot \tau_{Ed.web}^2} = 389.284 \cdot MPa$

Stiffeners

Stiffener over

$W_{y.st.o} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1} = 0.079 \cdot m^3$

$\sigma_{x.Ed.st.o} := \frac{M_{Ed}}{W_{y.st.o}} = 172.536 \cdot MPa$

Stiffener under

$W_{y.st.u} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1 - d_2} = -1.392 \cdot m^3$

$\sigma_{x.Ed.st.u} := \frac{M_{Ed}}{W_{y.st.u}} = -9.824 \cdot MPa$

Flange

$W_{y.fl} := \frac{I_y}{\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}} = 0.038 \cdot m^3$

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{W_{y.fl}} = 359.454 \cdot MPa$

$S_{tp} := \frac{t_{\ddot{o}f}}{2} \cdot b_{\ddot{o}f} \cdot \left(\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}\right) = 6.899 \times 10^{-3} \cdot m^3$

$\tau_{Ed.tp.mid} := \frac{V_{Ed.mid} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0 \cdot MPa$

$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0.672 \cdot MPa$

Calculation of load amplifiers, $\alpha_{ult,k}$, with von mises stress taken from sofistik.

-Formula for von mises in the denominator in $\alpha_{ult.k}$

$\alpha_{ult.k_mises} := \frac{f_{y2}}{\sigma_V} = 0.997$ σ_V from sofistik EC3-1-5 Ch.10(4) eq.10.3

Global slenderness $\lambda_{p,mises}$

$\lambda_p := \sqrt{\frac{\alpha_{ult.k_mises}}{\alpha_{cr_fem}}} = 0.708$

Flange

Flange, calculation of $\sigma_{cr,p}$:

$\psi_{flange} := 1$ $k_{\sigma.flange} := 0.43$ Cross sectional parts in compression with free edge SS-EN 1993-1-5 table 4.2

$\sigma_{E.flange} := \frac{\pi^2 \cdot E \cdot t_{uf}^2}{12 \cdot (1 - \nu^2) \cdot \left(\frac{b_{uf} - t_{web}}{2}\right)^2} = 1.58 \times 10^3 \cdot MPa$

$\sigma_{cr.p.fl} := k_{\sigma.flange} \cdot \sigma_{E.flange} = 679.428 \cdot MPa$

Flange; calculation of elastic critical column buckling stress $\sigma_{cr,c}$:

$\sigma_{cr.c.fl} := \frac{\pi^2 \cdot E \cdot t_{\ddot{o}f}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 1.493 \cdot MPa$ Unstiffened plate

Stiffener; calculation of $\sigma_{cr,p}$:

$\psi_{st} := 1$

$k_{\sigma.st} := 0.43$

$$\sigma_{E.st.1} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1-\nu^2) \cdot (b_{1.avst})^2} = 683.28 \cdot MPa$$ Part 1 stiffener

$$\sigma_{E.st.2} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1-\nu^2) \cdot (b_{2.avst})^2} = 3.374 \times 10^3 \cdot MPa$$ Part 2 stiffener

$$\sigma_{cr.p.st.1} := k_{\sigma.st} \cdot \sigma_{E.st.1} = 293.811 \cdot MPa$$

$$\sigma_{cr.p.st.2} := k_{\sigma.st} \cdot \sigma_{E.st.2} = 1.451 \times 10^3 \cdot MPa$$

Stiffener; calculation of elastic critical column buckling stress $\sigma_{cr.c}$:

$$\sigma_{cr.c.st} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1-\nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$$ Unstiffened plate

Calculation of the reduction factor χ_w for shear stess:

$\eta := 1.2$ EC3-1-5 Ch.5.1

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{0.7 + \lambda_p} & \text{otherwise} \end{cases}$$ EC3-1-5 tab.5.1

$\chi_w = 1.173$

WEB

Web; calculation of χ_{cx} (that depends on λ_p)

$\alpha := 0.49$

$e_2 := y_{tp.sl.1} - \frac{t_{web}}{2} = 50.81 \cdot mm$

$e_1 := C_y - e_2 - \frac{t_{web}}{2} = 14.018 \cdot mm$

$e := \max(e_1, e_2)$ EC3-1-5 4.5.3(5)

$i := \sqrt{\frac{I_{z.sl.1}}{A_{sl.1}}} = 0.077\,m$ EC3-1-5 4.5.3(5)

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.55$ EC3-1-5 4.5.3(5), stiffened plates

$\Phi_c := 0.5 \cdot \left[1 + \alpha_c \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.89$

$$\chi_{cx} := \min\left(\frac{1}{\Phi_c + \sqrt{\Phi_c^2 - \lambda_p^2}}, 1\right) = 0.699$$ EC3-1-1 eq. 6.49

Web; Calculation reduction factors ρ_p

Internal

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max[(3+\psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$ SS-en 1993-1-5(eq4.2) Internal cross sectional parts

$\psi_{liv} := \psi_{neg}(t_{uf}, t_{uf} + h_{liv}) = -1.055$ Stress distribution, $\psi_x = \sigma_2/\sigma_1$

$\rho_{px} := \rho_{4.2}(\lambda_p, \psi_{liv}) = 1$

Calculation of amplifers, $\sigma_{cr.p}$ and $\sigma_{cr.c}$, for column like buckling in x direction for the plate

$b_2 := d_2$ $b := b_1 + b_2$

$$a_c := 4.33 \cdot \sqrt[4]{\frac{I_{z.sl.1} \cdot b_1^2 \cdot b_2^2}{t_{web}^3 \cdot b}} = 5.127\,m$$

$$\sigma_{cr.sl.p} := \begin{cases} \left(\frac{1.05 \cdot E}{A_{sl.1}} \sqrt{\frac{I_{z.sl.1} \cdot t_{web}^3 \cdot b}{b_1 \cdot b_2}}\right) & \text{if } s_{ts} \ge a_c \\ \left[\frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} + \frac{E \cdot t_{web}^3 \cdot b \cdot s_{ts}^2}{4 \cdot \pi^2 \cdot (1-\nu^2) \cdot A_{sl.1} \cdot b_1^2 \cdot b_2^2}\right] & \text{if } s_{ts} \le a_c \end{cases}$$ Eq. A.4

$b_c := t_{uf} + h_{liv} - z_{tp} = 0.973\,m$

$b_{sl.1} := b_c - d_1 = 0.473\,m$

$b_t := h_{liv} - b_c = 1.027\,m$

$\sigma_{cr.p} := \sigma_{cr.sl.p} \cdot \frac{b_c}{b_{sl.1}} = 1.899 \times 10^3 \cdot MPa$ Stiffened plate with one longitudinal stiffener in the compression zone. EC3-1-5 Ch. A.2.2

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} = 153.121 \cdot MPa$ Stiffened plate. EC3-1-5 Ch. 4.5.3

$\sigma_{cr.c} := \sigma_{cr.sl} \cdot \frac{b_c}{b_{sl.1}} = 314.961 \cdot MPa$

Interaction plate like/column like

$\xi_x := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.web} := (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} = 1$

$$\rho_{cx.web} := \begin{cases} (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} \ge 1 \end{cases} = 1$$ According to EC1993-1-5 eq.10.1

Transverse stresses (z-direction), calculation of ρ_{pz}:

$\lambda_{p0} := 0.8$ $\alpha_p := 0.34$ Table B.1

$\varphi_p := 0.5 \cdot \left[1 + \alpha_p \cdot (\lambda_p - \lambda_{p0}) + \lambda_p\right] = 0.838$

$$\rho_{p.z} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.184 - 0.102i$$ EC1993-1-5, eq. B.1

Elastic critical buckling stress and the buckling stress for a planet that is not stiffened, seen from the z-direction:

$$\sigma_{cr.c.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1-\nu^2) \cdot h_{liv}^2} = 6.833 \cdot MPa$$

$$\sigma_{E.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1-\nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$$

$k_{\sigma z} := 4$

$\sigma_{cr.p.z} := k_{\sigma z} \cdot \sigma_{E.z} = 1.376 \cdot MPa$

Column buckling, χ_{cz}:

$\alpha_{unstiff} := 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.z} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot(\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.804$ Use λ_p according to EC3-1-5 Chapter 10

$\chi_{cz} := \dfrac{1}{\Phi_{c.z} + \sqrt{\Phi_{c.z}^{\,2} - \lambda_p^{\,2}}} = 0.844$ EC3-1-1 eq. 6.49

Interaction plate like/column like:

$\xi_z := \max\left(\min\left(\dfrac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 1\right), 0\right) = 0$

$\rho_{cz.web} := (\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz} = 0.844$

$\rho_{cz.web} := \begin{cases} (\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz} & \text{if } \dfrac{\left[(\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) \ldots\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \dfrac{\left[(\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} \geq 1 \end{cases} = 0.84$ According to EC1993-1-5 eq.10.1

The verification formula, web, with σ_{xEd}, σ_{zEd} and τ_{Ed} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots \leq 1 = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

Verification with V-factor according to Zizza

$c_{web} := \min\left(1.25 - 0.25\cdot\psi_{liv}, 1.25\right) = 1.25$

$\rho_{web.x.V} := \min\left[c_{web}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 1$

$\rho_{c.web.x.V} := (\rho_{web.x.V} - \chi_{cx})\cdot\xi_x\cdot(2 - \xi_x) + \chi_{cx} = 1$

$\xi_{z.z} := \max\left(\dfrac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 0\right) = 0$

$f := \lambda_p^{\,-0.5}\cdot\ln\left(\xi_{z.z} + 1\right)^{0.9} = 0$

$\rho_{c.web.z.V} := \chi_{cz} + (\rho_{p.z} - \chi_{cz})\cdot f = 0.844$

$V_{web} := \rho_{c.web.x.V}\cdot\rho_{c.web.z.V} = 0.844$

$e_3 := 1.25 + 0.75\cdot\chi_w^{\,2} = 2.281$

The verification formula, web, V factor according to Zizza.

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} \leq 1 = 1$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} = 0.987$$

Stiffeners

Reduction factors, cross sectional parts with free edge

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$ SS-EN 1993-1-5 eq 4.3 Cross sectional parts with free edge

$\psi := 1$

$\rho_{px.st.1} := \rho_{4.3}(\lambda_p, \psi) = 1$

Column buckling, $\chi_{cx.ft}$:

$\alpha_{unstiff} = 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.x.st.1} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot(\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.804$

$\chi_{cx.st.1} := \dfrac{1}{\Phi_{c.x.st.1} + \sqrt{\Phi_{c.x.st.1}^{\,2} - \lambda_p^{\,2}}} = 0.844$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$\xi_{x.st.1} := \max\left(\min\left(\dfrac{\sigma_{cr.p.st.1}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.st.1} := (\rho_{px.st.1} - \chi_{cx.st.1})\cdot\xi_{x.st.1}\cdot(2 - \xi_{x.st.1}) + \chi_{cx.st.1} = 1$ EC1993-1-5 eq.4.13

$\xi_{x.st.2} := \max\left(\min\left(\dfrac{\sigma_{cr.p.st.2}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$

The verification formula, stiffeners, with σ_{xEd} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \leq 1 = 1 \qquad \left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 = 0.206$$

Flange

Reduction factor, cross sectional parts with free edge

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$ SS-en 1993-1-5(eq4.3) cross sectional parts with free edge

$\psi := 1$

$\rho_{px.fl} := \rho_{4.3}(\lambda_p, \psi) = 1$

Column buckling, $\chi_{cx.fl}$:

$\alpha_{unstiff} = 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.x.fl} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.804$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\chi_{cx.fl} := \dfrac{1}{\Phi_{c.x.fl} + \sqrt{\Phi_{c.x.fl}^{\,2} - \lambda_p^{\,2}}} = 0.844$ Use λ_p, see EC3-1-5 Ch. 10

EC3-1-1 eq. 6.49

Interaction plate like/column like

$\xi_{x.fl} := \max\left(\min\left(\dfrac{\sigma_{cr.p.fl}}{\sigma_{cr.c.fl}} - 1, 1\right), 0\right) = 1$ Platelike behaviour

$\rho_{cx.fl} := \left(\rho_{px.fl} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_{x.fl}\right) + \chi_{cx.fl} = 1$ EC1993-1-5 eq.4.13

$\zeta_3 := \max\left(1.25 + 0.75\cdot\chi_w^{\,2}, 2\right) = 2.281$

The verification formula, flange:

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 \le 1 = 1$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 = 0.997$$

The verification formula, flange, V factor according to Zizza.

$\psi_{tp} := 1$ Uniform compression

$c_{fl} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

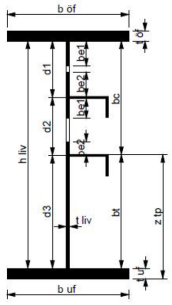
$\rho_{fl.x.V} := \min\left[c_{fl}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.974$

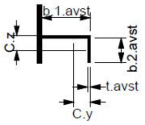
$\rho_{c.fl.x.V} := \left(\rho_{fl.x.V} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_x\right) + \chi_{cx.fl} = 0.974$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} \le 1 = 0$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.tp}}{\left(\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}\right)}\right]^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} = 1.025$$

Calculation of a longitudinal stiffened I-beam with the RSM - 250mm horizontal stiffener





Material parameters:

$\gamma_M := 1$

$E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$f_y := \begin{pmatrix} f_y(16mm) \\ f_y(40mm) \\ f_y(41mm) \end{pmatrix} = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Area

$A_{öf} := t_{öf} \cdot b_{öf}$ $A_{liv} := t_{web} \cdot h_{liv}$ $A_{uf} := t_{uf} \cdot b_{uf}$

$A_{avst} := t_{avst} \cdot (b_{1.avst} + b_{2.avst})$ $A_{tot} := A_{öf} + A_{liv} + A_{uf} + 2 \cdot A_{avst} = 6.016 \times 10^4 \cdot mm^2$

Change of centre of gravity in y- and z-direction

$$C_y := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{b_{1.avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right)}{A_{avst}} = 93.5 \cdot mm$$

$$C_z := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(t_{avst} + \frac{b_{2.avst}}{2}\right)}{A_{avst}} = 19.5 \cdot mm$$

Geometry

$t_{öf} := 25mm$

$b_{öf} := 560mm$

$t_{web} := 12mm$

$h_{liv} := 2m$

$d_1 := 0.5m$

$d_2 := 0.5m$

$d_3 := 1m$

$t_{uf} := 25mm$

$b_{uf} := 560mm$

Horizontal stiffeners:

$b_{1.avst} := 250mm$

$b_{2.avst} := 90mm$

$t_{avst} := 12mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

Number of longitudinal stiffeners:

$n := 2$

Poissons ratio

$\nu := 0.3$

Distance from bottom fibre to centre of gravity

$$z_{tp} := \frac{A_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + A_{liv} \cdot \left(t_{uf} + \frac{h_{liv}}{2}\right) + A_{öf} \cdot \left(t_{uf} + h_{liv} + \frac{t_{öf}}{2}\right) \ldots}{+ A_{avst} \cdot (t_{uf} + d_2 + d_3 - C_z) + A_{avst} \cdot (t_{uf} + d_3 - C_z)} = 1.056\,m$$

$$A_{tot}$$

Moment of inirtia, stiffeners $I_{z.avst}$

$$I_{z.avst} := \frac{b_{2.avst} \cdot t_{avst}^3}{12} + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2} - C_y\right)^2 \ldots = 2.688 \times 10^{-5} m^4$$

$$+ \frac{t_{avst} \cdot b_{1.avst}^3}{12} + t_{avst} \cdot b_{1.avst} \cdot \left(\frac{b_{1.avst}}{2} - C_y\right)^2$$

The stiffeners area and moment of inertia fig A.1

$b_1 := d_1$

$b_{3.c} := t_{uf} + d_3 + d_2 - z_{tp} = 0.469\,m$

Highest stress(under the flange) is set to the yield stress

$$\sigma_{max} := f_y = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Stress at a z-koord, bottom flange in tenstion

$$\sigma_{neg}(z) := \left(\frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}\right) \cdot -1$$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$$

$\psi_{liv.1} := \psi_{neg}(t_{uf} + d_2 + d_3, t_{uf} + h_{liv}) = 0.447$

$\psi_{liv.2} := \psi_{neg}(t_{uf} + d_3, t_{uf} + d_2 + d_3) = -0.067$

$\psi_1 := \psi_{liv.1} = 0.447$

$\psi_3 := \psi_{liv.2} = -0.067$

$b_{1.inf} := \frac{3 - \psi_1}{5 - \psi_1} \cdot b_1 = 280.382 \cdot mm$

$b_{3.sup} := 0.4 \cdot b_{3.c} = 187.494 \cdot mm$

$A_{sl.1} := A_{avst} + t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) = 9.839 \times 10^{-3}\, m^2$

$y_{tp.sl.1} := \frac{t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot \frac{t_{web}}{2} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web})}{A_{sl.1}} = 73.388 \cdot mm$

Stiffeners moment of inertia out from the web plate

$I_{z.sl.1} := I_{z.avst} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web} - y_{tp.sl.1})^2 \ldots + \frac{(b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web}^3}{12} \ldots + (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web} \cdot \left(\frac{t_{web}}{2} - y_{tp.sl.1}\right)^2 = 9.001 \times 10^{-5}\, m^4$

Moment of inertia, Von mises stress and alpha critical

$q_d := 154 \frac{kN}{m}$ $I_y := 0.0376 m^4$ From EWM Ramboll

$\sigma_{v.unit} := \frac{2.34 MPa}{\frac{kN}{m}}$ From FEM, close to middle of the span in top flange

$\sigma_V := \sigma_{v.unit} \cdot q_d = 360.36 \cdot MPa$

$\alpha_{cr.UNIT} := 338.34 \cdot \frac{kN}{m}$ Eigenvalue from FEM SOFiSTiK

$\alpha_{cr_fem} := \frac{\alpha_{cr.UNIT}}{q_d} = 2.197$

Load global

$V_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)}{2} = 2.059 \times 10^3 \cdot kN$

$M_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)^2}{8} = 1.377 \times 10^4 \cdot kN \cdot m$

$V_{Ed.mid} := 0$

Web

$W_{y.web} := \frac{I_y}{h_{liv} + t_{uf} + t_{\ddot{o}f} - z_{tp}} = 0.038 \cdot m^3$

$\sigma_{x.Ed.web} := \frac{M_{Ed}}{W_{y.web}} = 363.832 \cdot MPa$

$\sigma_{z.Ed.web} := \frac{q_d}{t_{web}} = 12.833 \cdot MPa$

$S := t_{\ddot{o}f} \cdot b_{\ddot{o}f} \cdot \left(t_{uf} + h_{liv} + \frac{t_{\ddot{o}f}}{2} - z_{tp}\right) + \frac{h_{liv}}{2} \cdot t_{web} \cdot \left(t_{uf} + \frac{h_{liv}}{2} + \frac{h_{liv}}{4} - z_{tp}\right) = 0.019 \cdot m^3$

$\tau_{Ed.web} := \frac{V_{Ed} \cdot S}{I_y \cdot t_{web}} = 88.363 \cdot MPa$

$\tau_{Ed.web.2mfromsupport} := \frac{(V_{Ed} - q_d \cdot 2m) \cdot S}{I_y \cdot t_{web}} = 75.146 \cdot MPa$

Calculation of .von mises stess analyttical

$\sigma_{v.analytical} := \sqrt{\sigma_{x.Ed.web}^2 + \sigma_{z.Ed.web}^2 - \sigma_{x.Ed.web} \cdot \sigma_{z.Ed.web} + 3 \cdot \tau_{Ed.web}^2} = 388.965 \cdot MPa$

Stiffeners

Stiffener over

$W_{y.st.o} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1} = 0.08 \cdot m^3$

$\sigma_{x.Ed.st.o} := \frac{M_{Ed}}{W_{y.st.o}} = 171.616 \cdot MPa$

Stiffener under

$W_{y.st.u} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1 - d_2} = -1.203 \cdot m^3$

$\sigma_{x.Ed.st.u} := \frac{M_{Ed}}{W_{y.st.u}} = -11.447 \cdot MPa$

Flange

$W_{y.fl} := \frac{I_y}{\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}} = 0.038 \cdot m^3$

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{W_{y.fl}} = 359.256 \cdot MPa$

$S_{tp} := \frac{t_{\ddot{o}f}}{2} \cdot b_{\ddot{o}f} \cdot \left(\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}\right) = 6.869 \times 10^{-3} \cdot m^3$

$\tau_{Ed.tp.mid} := \frac{V_{Ed.mid} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0 \cdot MPa$

$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0.672 \cdot MPa$

Calculation of load amplifiers, $\alpha_{ult.k}$, with von mises stress taken from sofistik.

-Formula for von mises in the denominator in α_{ultk}

$\alpha_{ult.k_mises} := \frac{f_{y2}}{\sigma_V} = 0.999$ σ_V from sofistik EC3-1-5 Ch.10(4) eq.10.3

Global slenderness $\lambda_{p,mises}$

$\lambda_p := \sqrt{\frac{\alpha_{ult.k_mises}}{\alpha_{cr_fem}}} = 0.674$

Flange

Flange, calculation of $\sigma_{cr.p}$:

$\psi_{flange} := 1$ $k_{\sigma.flange} := 0.43$ Cross sectional parts in compression with free edge SS-EN 1993-1-5 table 4.2

$\sigma_{E.flange} := \frac{\pi^2 \cdot E \cdot t_{uf}^2}{12 \cdot (1 - \nu^2) \cdot \left(\frac{b_{uf} - t_{web}}{2}\right)^2} = 1.58 \times 10^3 \cdot MPa$

$\sigma_{cr.p.fl} := k_{\sigma.flange} \cdot \sigma_{E.flange} = 679.428 \cdot MPa$

Flange; calculation of elastic critical column buckling stress $\sigma_{cr.c}$:

$\sigma_{cr.c.fl} := \frac{\pi^2 \cdot E \cdot t_{\ddot{o}f}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 1.493 \cdot MPa$ Unstiffened plate

Stiffener; calculation of $\sigma_{cr,p}$:

$\psi_{st} := 1$

$k_{\sigma.st} := 0.43$

$\sigma_{E.st.1} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{1.avst})^2} = 437.299 \cdot MPa$ Part 1 stiffener

$\sigma_{E.st.2} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{2.avst})^2} = 3.374 \times 10^3 \cdot MPa$ Part 2 stiffener

$\sigma_{cr.p.st.1} := k_{\sigma.st} \cdot \sigma_{E.st.1} = 188.039 \cdot MPa$

$\sigma_{cr.p.st.2} := k_{\sigma.st} \cdot \sigma_{E.st.2} = 1.451 \times 10^3 \cdot MPa$

Stiffener; calculation of elastic critical column buckling stress $\sigma_{cr,c}$:

$\sigma_{cr.c.st} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$ Unstiffened plate

Calculation of the reduction factor χ_w for shear stess:

$\eta := 1.2$ EC3-1-5 Ch.5.1

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{0.7 + \lambda_p} & \text{otherwise} \end{cases}$$ EC3-1-5 tab.5.1

$\chi_w = 1.2$

WEB

Web; calculation of χ_{cx} (that depends on λ_p)

$\alpha := 0.49$

$e_2 := y_{tp.sl.1} - \frac{t_{web}}{2} = 67.388 \cdot mm$

$e_1 := C_y - e_2 - \frac{t_{web}}{2} = 20.112 \cdot mm$

$\underset{\sim}{e} := \max(e_1, e_2)$ EC3-1-5 4.5.3(5)

$i := \sqrt{\frac{I_{z.sl.1}}{A_{sl.1}}} = 0.096\,m$ EC3-1-5 4.5.3(5)

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.553$ EC3-1-5 4.5.3(5), stiffened plates

$\Phi_c := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.859$

$\chi_{cx} := \min\left(\frac{1}{\Phi_c + \sqrt{\Phi_c^2 - \lambda_p^2}}, 1\right) = 0.719$ EC3-1-1 eq. 6.49

Web; Calculation reduction factors ρ_p

Internal

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$ SS-EN 1993-1-5(eq4.2) Internal cross sectional parts

$\psi_{liv} := \psi_{neg}(t_{uf}, t_{uf} + h_{liv}) = -1.065$ Stress distribution, $\psi_x = \sigma_2/\sigma_1$

$\rho_{px} := \rho_{4.2}(\lambda_p, \psi_{liv}) = 1$

Calculation of amplifers, $\sigma_{cr,p}$ and $\sigma_{cr,c}$, for column like buckling in x direction for the plate

$b_2 := d_2 \qquad b := b_1 + b_2$

$a_c := 4.33 \cdot \sqrt[4]{\frac{I_{z.sl.1} \cdot b_1^2 \cdot b_2^2}{t_{web}^3 \cdot b}} = 5.816\,m$

$$\sigma_{cr.sl.p} := \begin{cases} \left(\frac{1.05 \cdot E}{A_{sl.1}} \cdot \sqrt{\frac{I_{z.sl.1} \cdot t_{web}^3 \cdot b}{b_1 \cdot b_2}}\right) & \text{if } s_{ts} \ge a_c \\ \left[\frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} + \frac{E \cdot t_{web}^3 \cdot b \cdot s_{ts}^2}{4 \cdot \pi^2 \cdot (1 - \nu^2) \cdot A_{sl.1} \cdot b_1^2 \cdot b_2^2}\right] & \text{if } s_{ts} \le a_c \end{cases}$$ Eq. A.4

$b_c := t_{uf} + h_{liv} - z_{tp} = 0.969\,m$

$b_{sl.1} := b_c - d_1 = 0.469\,m$

$b_t := h_{liv} - b_c = 1.031\,m$

$\sigma_{cr.p} := \sigma_{cr.sl.p} \cdot \frac{b_c}{b_{sl.1}} = 2.311 \times 10^3 \cdot MPa$ Stiffened plate with one longitudinal stiffener in the compression zone. EC3-1-5 Ch. A.2.2

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} = 238.64 \cdot MPa$ Stiffened plate. EC3-1-5 Ch. 4.5.3

$\sigma_{cr.c} := \sigma_{cr.sl} \cdot \frac{b_c}{b_{sl.1}} = 493.197 \cdot MPa$

Interaction plate like/column like

$\xi_x := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.web} := (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} = 1$

$$\rho_{cx.web} := \begin{cases} (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} & \text{if } \frac{[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}] \cdot \alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \frac{[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}] \cdot \alpha_{ult.k_mises}}{\gamma_M} \ge 1 \end{cases} = 1$$ According to EC1993-1-5 eq.10.1

Transverse stresses (z-direction), calculation of ρ_{pz}:

$\lambda_{p0} := 0.8 \qquad \alpha_p := 0.34$ Table B.1

$\varphi_p := 0.5 \cdot \left[1 + \alpha_p \cdot (\lambda_p - \lambda_{p0}) + \lambda_p\right] = 0.816$

$$\rho_{p.z} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.21 - 0.139i$$ EC1993-1-5, eq. B.1

Elastic critical buckling stress and the buckling stress for a planet that is not stiffened, seen from the z-direction:

$\sigma_{cr.c.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1 - \nu^2) \cdot h_{liv}^2} = 6.833 \cdot MPa$

$\sigma_{E.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$

$k_{\sigma z} := 4$

$\sigma_{cr.p.z} := k_{\sigma z} \cdot \sigma_{E.z} = 1.376 \cdot MPa$

Column buckling, χ_{cz}:

$\alpha_{unstiff} := 0.21$ — EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.z} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.777$ — Use λ_p according to EC3-1-5 Chapter 10

$\chi_{cz} := \dfrac{1}{\Phi_{c.z} + \sqrt{\Phi_{c.z}^{\,2} - \lambda_p^{\,2}}} = 0.859$ — EC3-1-1 eq. 6.49

Interaction plate like/column like:

$\xi_z := \max\left(\min\left(\dfrac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 1\right), 0\right) = 0$

$\rho_{cz.web} := \left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz} = 0.859$

$\rho_{cz.web} := \begin{cases} \left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz} & \text{if } \dfrac{\left[\left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) \ldots\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} \geq 1 \end{cases} = 0.85$ — According to EC1993-1-5 eq.10.1

The verification formula, web, with σ_{xEd}, σ_{zEd} and τ_{Ed} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots \leq 1 = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots = 0.993$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

Verification with V-factor according to Zizza

$c_{web} := \min\left(1.25 - 0.25\cdot\psi_{liv}, 1.25\right) = 1.25$

$\rho_{web.x.V} := \min\left[c_{web}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 1$

$\rho_{c.web.x.V} := \left(\rho_{web.x.V} - \chi_{cx}\right)\cdot\xi_x\cdot\left(2 - \xi_x\right) + \chi_{cx} = 1$

$\xi_{z.z} := \max\left(\dfrac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 0\right) = 0$

$f := \lambda_p^{\,-0.5}\cdot\ln\left(\xi_{z.z} + 1\right)^{0.9} = 0$

$\rho_{c.web.z.V} := \chi_{cz} + \left(\rho_{p.z} - \chi_{cz}\right)\cdot f = 0.859$

$V_{web} := \rho_{c.web.x.V}\cdot\rho_{c.web.z.V} = 0.859$

$e_3 := 1.25 + 0.75\cdot\chi_w^{\,2} = 2.33$

The verification formula, web, V factor according to Zizza.

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} \leq 1 = 1$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} = 0.982$$

Stiffeners

Reduction factors, cross sectional parts with free edge

$\rho_{4.3}\left(\lambda_p, \psi\right) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$ — SS-EN 1993-1-5 eq 4.3 Cross sectional parts with free edge

$\psi := 1$

$\rho_{px.st.1} := \rho_{4.3}\left(\lambda_p, \psi\right) = 1$

Column buckling, $\chi_{cx.fl}$:

$\alpha_{unstiff} = 0.21$ — EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.x.st.1} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.777$ — Use λ_p EC3-1-5 Ch. 10

$\chi_{cx.st.1} := \dfrac{1}{\Phi_{c.x.st.1} + \sqrt{\Phi_{c.x.st.1}^{\,2} - \lambda_p^{\,2}}} = 0.859$ — EC3-1-1 eq. 6.49

Interaktion plate like/column like

$\xi_{x.st.1} := \max\left(\min\left(\dfrac{\sigma_{cr.p.st.1}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.st.1} := \left(\rho_{px.st.1} - \chi_{cx.st.1}\right)\cdot\xi_{x.st.1}\cdot\left(2 - \xi_{x.st.1}\right) + \chi_{cx.st.1} = 1$ — EC1993-1-5 eq.4.13

$\xi_{x.st.2} := \max\left(\min\left(\dfrac{\sigma_{cr.p.st.2}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$

The verification formula, stiffeners, with σ_{xEd} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \leq 1 = 1 \qquad \left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 = 0.204$$

Flange

Reduction factor, cross sectional parts with free edge

$\rho_{4.3}\left(\lambda_p, \psi\right) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$ — SS-en 1993-1-5(eq4.3) cross sectional parts with free edge

$\psi := 1$

$\rho_{px.fl} := \rho_{4.3}\left(\lambda_p, \psi\right) = 1$

Column buckling, $\chi_{cx.fl}$:

$\alpha_{unstiff} = 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.x.fl} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.777$ Use λ_p, see EC3-1-5 Ch. 10

$\chi_{cx.fl} := \dfrac{1}{\Phi_{c.x.fl} + \sqrt{\Phi_{c.x.fl}^{\,2} - \lambda_p^{\,2}}} = 0.859$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$\xi_{x.fl} := \max\left(\min\left(\dfrac{\sigma_{cr.p.fl}}{\sigma_{cr.c.fl}} - 1, 1\right), 0\right) = 1$ Platelike behaviour

$\rho_{cx.fl} := \left(\rho_{px.fl} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_{x.fl}\right) + \chi_{cx.fl} = 1$ EC1993-1-5 eq.4.13

$\zeta_3 := \max\left(1.25 + 0.75\cdot\chi_w^{\,2}, 2\right) = 2.33$

The verification formula, flange:

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 \le 1 = 1$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 = 0.996$$

The verification formula, flange, V factor according to Zizza.

$\psi_{tp} := 1$ Uniform compression

$c_{fl} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

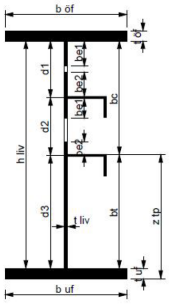
$\rho_{fl.x.V} := \min\left[c_{fl}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.999$

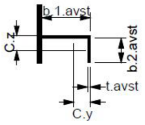
$\rho_{c.fl.x.V} := \left(\rho_{fl.x.V} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_x\right) + \chi_{cx.fl} = 0.999$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} \le 1 = 1$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.tp}}{\left(\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}\right)}\right]^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} = 0.999$$

Calculation of a longitudinal stiffened I-beam with the RSM - 300mm horizontal stiffener





Geometry

$t_{öf} := 25mm$

$b_{öf} := 560mm$

$t_{web} := 12mm$

$h_{liv} := 2m$

$d_1 := 0.5m$

$d_2 := 0.5m$

$d_3 := 1m$

$t_{uf} := 25mm$

$b_{uf} := 560mm$

Horizontal stiffeners:

$b_{1.avst} := 300mm$

$b_{2.avst} := 90mm$

$t_{avst} := 12mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

Number of longitudinal stiffeners:

$n := 2$

Poissons ratio

$\nu := 0.3$

Material parameters:

$\gamma_M := 1$

$E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$f_{yk} := \begin{pmatrix} f_y(16mm) \\ f_y(40mm) \\ f_y(41mm) \end{pmatrix} = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Area

$A_{öf} := t_{öf} \cdot b_{öf}$ $A_{liv} := t_{web} \cdot h_{liv}$ $A_{uf} := t_{uf} \cdot b_{uf}$

$A_{avst} := t_{avst} \cdot (b_{1.avst} + b_{2.avst})$ $A_{tot} := A_{öf} + A_{liv} + A_{uf} + 2 \cdot A_{avst} = 6.136 \times 10^{4} \cdot mm^2$

Change of centre of gravity in y- and z-direction

$$C_y := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{b_{1.avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right)}{A_{avst}} = 116.769 \cdot mm$$

$$C_z := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(t_{avst} + \frac{b_{2.avst}}{2}\right)}{A_{avst}} = 17.769 \cdot mm$$

Distance from bottom fibre to centre of gravity

$$z_{tp} := \frac{\begin{matrix} A_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + A_{liv} \cdot \left(t_{uf} + \frac{h_{liv}}{2}\right) + A_{öf} \cdot \left(t_{uf} + h_{liv} + \frac{t_{öf}}{2}\right) \dots \\ + A_{avst} \cdot (t_{uf} + d_2 + d_3 - C_z) + A_{avst} \cdot (t_{uf} + d_3 - C_z) \end{matrix}}{A_{tot}} = 1.06\,m$$

Moment of inirtia, stiffeners $I_{z.avst}$

$$I_{z.avst} := \frac{b_{2.avst} \cdot t_{avst}^3}{12} + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2} - C_y\right)^2 \dots + \frac{t_{avst} \cdot b_{1.avst}^3}{12} + t_{avst} \cdot b_{1.avst} \cdot \left(\frac{b_{1.avst}}{2} - C_y\right)^2 = 4.424 \times 10^{-5}\,m^4$$

The stiffeners area and moment of inertia fig A.1

$b_1 := d_1$

$b_{3.c} := t_{uf} + d_3 + d_2 - z_{tp} = 0.465\,m$

Highest stress(under the flange) is set to the yield stress

$$\sigma_{max} := f_y = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Stress at a z-koord, bottom flange in tenstion

$$\sigma_{neg}(z) := \left(\frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}\right) \cdot -1$$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$$

$\psi_{liv.1} := \psi_{neg}(t_{uf} + d_2 + d_3, t_{uf} + h_{liv}) = 0.445$

$\psi_{liv.2} := \psi_{neg}(t_{uf} + d_3, t_{uf} + d_2 + d_3) = -0.076$

$\psi_1 := \psi_{liv.1} = 0.445$

$\psi_3 := \psi_{liv.2} = -0.076$

$$b_{1.inf} := \frac{3 - \psi_1}{5 - \psi_1} \cdot b_1 = 280.481 \cdot mm$$

$$b_{3.sup} := 0.4 \cdot b_{3.c} = 185.83 \cdot mm$$

$$A_{sl.1} := A_{avst} + t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) = 0.01\,m^2$$

$$y_{tp.sl.1} := \frac{t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot \frac{t_{web}}{2} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web})}{A_{sl.1}} = 90.993 \cdot mm$$

Stiffeners moment of inertia out from the web plate

$$I_{z.sl.1} := I_{z.avst} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web} - y_{tp.sl.1})^2 \ldots + \frac{(b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web}^3}{12} \ldots + (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web} \cdot \left(\frac{t_{web}}{2} - y_{tp.sl.1}\right)^2 = 1.366 \times 10^{-4}\,m^4$$

Moment of inertia, Von mises stress and alpha critical

$q_d := 155 \frac{kN}{m}$ $I_y := 0.0377 m^4$ From EWM Ramboll

$\sigma_{v.unit} := \frac{2.32MPa}{\frac{kN}{m}}$ From FEM, close to middle of the span in top flange

$$\sigma_V := \sigma_{v.unit} \cdot q_d = 359.6 \cdot MPa$$

$\alpha_{cr.UNIT} := 341.19 \cdot \frac{kN}{m}$ Eigenvalue from FEM SOFiSTiK

$$\alpha_{cr_fem} := \frac{\alpha_{cr.UNIT}}{q_d} = 2.201$$

Load global

$$V_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)}{2} = 2.073 \times 10^3 \cdot kN$$

$$M_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)^2}{8} = 1.386 \times 10^4 \cdot kN \cdot m$$

$$V_{Ed.mid} := 0$$

Web

$$W_{y.web} := \frac{I_y}{h_{liv} + t_{uf} + t_{\ddot{o}f} - z_{tp}} = 0.038 \cdot m^3$$

$$\sigma_{x.Ed.web} := \frac{M_{Ed}}{W_{y.web}} = 363.695 \cdot MPa$$

$$\sigma_{z.Ed.web} := \frac{q_d}{t_{web}} = 12.917 \cdot MPa$$

$$S := t_{\ddot{o}f} \cdot b_{\ddot{o}f} \cdot \left(t_{uf} + h_{liv} + \frac{t_{\ddot{o}f}}{2} - z_{tp}\right) + \frac{h_{liv}}{2} \cdot t_{web} \cdot \left(t_{uf} + \frac{h_{liv}}{2} + \frac{h_{liv}}{4} - z_{tp}\right) = 0.019 \cdot m^3$$

$$\tau_{Ed.web} := \frac{V_{Ed} \cdot S}{I_y \cdot t_{web}} = 88.205 \cdot MPa$$

$$\tau_{Ed.web.2mfromsupport} := \frac{(V_{Ed} - q_d \cdot 2m) \cdot S}{I_y \cdot t_{web}} = 75.011 \cdot MPa$$

Calculation of von mises stess analyttical

$$\sigma_{v.analytical} := \sqrt{\sigma_{x.Ed.web}^2 + \sigma_{z.Ed.web}^2 - \sigma_{x.Ed.web} \cdot \sigma_{z.Ed.web} + 3 \cdot \tau_{Ed.web}^2} = 388.694 \cdot MPa$$

Stiffeners

Stiffener over

$$W_{y.st.o} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1} = 0.081 \cdot m^3$$

$$\sigma_{x.Ed.st.o} := \frac{M_{Ed}}{W_{y.st.o}} = 170.743 \cdot MPa$$

Stiffener under

$$W_{y.st.u} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1 - d_2} = -1.064 \cdot m^3$$

$$\sigma_{x.Ed.st.u} := \frac{M_{Ed}}{W_{y.st.u}} = -13.02 \cdot MPa$$

Flange

$$W_{y.fl} := \frac{I_y}{\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}} = 0.039 \cdot m^3$$

$$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{W_{y.fl}} = 359.101 \cdot MPa$$

$$S_{tp} := \frac{t_{\ddot{o}f}}{2} \cdot b_{\ddot{o}f} \cdot \left(\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}\right) = 6.84 \times 10^{-3} \cdot m^3$$

$$\tau_{Ed.tp.mid} := \frac{V_{Ed.mid} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0 \cdot MPa$$

$$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0.671 \cdot MPa$$

Calculation of load amplifiers, $\alpha_{ult,k}$, with von mises stress taken from sofistik.

-Formula for von mises in the denominator in α_{ultk}

$\alpha_{ult.k_mises} := \frac{f_{y_2}}{\sigma_V} = 1.001$ σ_V from sofistik EC3-1-5 Ch.10(4) eq.10.3

Global slenderness $\lambda_{p,mises}$

$$\lambda_p := \sqrt{\frac{\alpha_{ult.k_mises}}{\alpha_{cr_fem}}} = 0.674$$

Flange

Flange, calculation of $\sigma_{cr,p}$:

$\psi_{flange} := 1$ $k_{\sigma.flange} := 0.43$

Cross sectional parts in compression with free edge SS-EN 1993-1-5 table 4.2

$$\sigma_{E.flange} := \frac{\pi^2 \cdot E \cdot t_{uf}^2}{12 \cdot (1 - \nu^2) \cdot \left(\frac{b_{uf} - t_{web}}{2}\right)^2} = 1.58 \times 10^3 \cdot MPa$$

$$\sigma_{cr.p.fl} := k_{\sigma.flange} \cdot \sigma_{E.flange} = 679.428 \cdot MPa$$

Flange; calculation of elastic critical column buckling stress $\sigma_{cr,c}$:

$$\sigma_{cr.c.fl} := \frac{\pi^2 \cdot E \cdot t_{\ddot{o}f}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 1.493 \cdot MPa$$

Unstiffened plate

Stiffener; calculation of $\sigma_{cr,p}$:

$\psi_{st} := 1$

$k_{\sigma.st} := 0.43$

$$\sigma_{E.st.1} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{1.avst})^2} = 303.68 \cdot MPa$$ Part 1 stiffener

$$\sigma_{E.st.2} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot (b_{2.avst})^2} = 3.374 \times 10^3 \cdot MPa$$ Part 2 stiffener

$\sigma_{cr.p.st.1} := k_{\sigma.st} \cdot \sigma_{E.st.1} = 130.582 \cdot MPa$

$\sigma_{cr.p.st.2} := k_{\sigma.st} \cdot \sigma_{E.st.2} = 1.451 \times 10^3 \cdot MPa$

Stiffener; calculation of elastic critical column buckling stress $\sigma_{cr.c}$:

$$\sigma_{cr.c.st} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$$ Unstiffened plate

Calculation of the reduction factor χ_w for shear stess:

$\eta := 1.2$ EC3-1-5 Ch.5.1

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{0.7 + \lambda_p} & \text{otherwise} \end{cases}$$ EC3-1-5 tab.5.1

$\chi_w = 1.2$

WEB

Web; calculation of χ_{cx} (that depends on λ_p)

$\alpha := 0.49$

$e_2 := y_{tp.sl.1} - \frac{t_{web}}{2} = 84.993 \cdot mm$

$e_1 := C_y - e_2 - \frac{t_{web}}{2} = 25.777 \cdot mm$

$e := \max(e_1, e_2)$ EC3-1-5 4.5.3(5)

$i := \sqrt{\frac{I_{z.sl.1}}{A_{sl.1}}} = 0.115\,m$ EC3-1-5 4.5.3(5)

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.557$ EC3-1-5 4.5.3(5), för stiffened plates

$\Phi_c := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.859$

$\chi_{cx} := \min\left(\frac{1}{\Phi_c + \sqrt{\Phi_c^2 - \lambda_p^2}}, 1\right) = 0.718$ EC3-1-1 eq. 6.49

Web; Calculation reduction factors ρ_p

Internal

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$ SS-en 1993-1-5(eq4.2) Tvärsnittsdelar inre

$\psi_{liv} := \psi_{neg}(t_{uf}, t_{uf} + h_{liv}) = -1.073$ Stress distribution, $\psi_x=\sigma_2/\sigma_1$

$\rho_{px} := \rho_{4.2}(\lambda_p, \psi_{liv}) = 1$

Calculation of amplifers, $\sigma_{cr.p}$ and $\sigma_{cr.c}$, for column like buckling in x direction for the plate

$b_2 := d_2$ $b := b_1 + b_2$

$$a_c := 4.33 \cdot \sqrt[4]{\frac{I_{z.sl.1} \cdot b_1^2 \cdot b_2^2}{t_{web}^3 \cdot b}} = 6.456\,m$$

$$\sigma_{cr.sl.p} := \begin{cases} \left(\frac{1.05 \cdot E}{A_{sl.1}} \cdot \sqrt{\frac{I_{z.sl.1} \cdot t_{web}^3 \cdot b}{b_1 \cdot b_2}}\right) & \text{if } s_{ts} \ge a_c \\ \left[\frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} + \frac{E \cdot t_{web}^3 \cdot b \cdot s_{ts}^2}{4 \cdot \pi^2 \cdot (1 - \nu^2) \cdot A_{sl.1} \cdot b_1^2 \cdot b_2^2}\right] & \text{if } s_{ts} \le a_c \end{cases}$$ Eq. A.4

$b_c := t_{uf} + h_{liv} - z_{tp} = 0.965\,m$

$b_{sl.1} := b_c - d_1 = 0.465\,m$

$b_t := h_{liv} - b_c = 1.035\,m$

$\sigma_{cr.p} := \sigma_{cr.sl.p} \cdot \frac{b_c}{b_{sl.1}} = 2.7 \times 10^3 \cdot MPa$ Stiffened plate with one longitudinal stiffener in the compression zone. EC3-1-5 Ch. A.2.2

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} = 342.009 \cdot MPa$ Stiffened plate. EC3-1-5 Ch. 4.5.3

$\sigma_{cr.c} := \sigma_{cr.sl} \cdot \frac{b_c}{b_{sl.1}} = 710.098 \cdot MPa$

Interaction plate like/column like

$\xi_x := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.web} := (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} = 1$

$$\rho_{cx.web} := \begin{cases} (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} \ge 1 \end{cases} = 1$$ According to EC1993-1-5 eq.10.1

Transverse stresses (z-direction), calculation of ρ_{pz}:

$\lambda_{p0} := 0.8$ $\alpha_p := 0.34$ Table B.1

$\varphi_p := 0.5 \cdot \left[1 + \alpha_p \cdot (\lambda_p - \lambda_{p0}) + \lambda_p\right] = 0.816$

$$\rho_{p.z} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.21 - 0.139i$$ EC1993-1-5, eq. B.1

Elastic critical buckling stress and the buckling stress for a planet that is not stiffened, seen from the z-direction:

$\sigma_{cr.c.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1 - \nu^2) \cdot h_{liv}^2} = 6.833 \cdot MPa$

$\sigma_{E.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$

$k_{\sigma z} := 4$

$\sigma_{cr.p.z} := k_{\sigma z} \cdot \sigma_{E.z} = 1.376 \cdot MPa$

Column buckling, χ_{cz}:

$\alpha_{unstiff} := 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.z} := 0.5 \cdot \left[1 + \alpha_{unstiff} \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.777$ Use λ_p according to EC3-1-5 Chapter 10

$$\chi_{cz} := \frac{1}{\Phi_{c.z} + \sqrt{\Phi_{c.z}^{2} - \lambda_p^{2}}} = 0.859$$ EC3-1-1 eq. 6.49

Interaction plate like/column like:

$$\xi_z := \max\left(\min\left(\frac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 1\right), 0\right) = 0$$

$$\rho_{cz.web} := (\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz} = 0.859$$

$$\rho_{cz.web} := \begin{cases} (\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz} & \text{if } \frac{\left[(\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) \ldots + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} < 1 \quad = 0.85 \\ 1 & \text{if } \frac{\left[(\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} \geq 1 \end{cases}$$

According to EC1993-1-5 eq.10.1

The verification formula, web, with σ_{xEd}, σ_{zEd} and τ_{Ed} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots \leq 1 = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots = 0.992$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

Verification with V-factor according to Zizza

$$c_{web} := \min\left(1.25 - 0.25\cdot\psi_{liv}, 1.25\right) = 1.25$$

$$\rho_{web.x.V} := \min\left[c_{web}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{2}}\right), 1\right] = 1$$

$$\rho_{c.web.x.V} := (\rho_{web.x.V} - \chi_{cx})\cdot\xi_x\cdot(2 - \xi_x) + \chi_{cx} = 1$$

$$\xi_{z.z} := \max\left(\frac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 0\right) = 0$$

$$f := \lambda_p^{-0.5\cdot\ln\left(\xi_{z.z} + 1\right)^{0.9}} = 0$$

$$\rho_{c.web.z.V} := \chi_{cz} + (\rho_{p.z} - \chi_{cz})\cdot f = 0.859$$

$$V_{web} := \rho_{c.web.x.V}\cdot\rho_{c.web.z.V} = 0.859$$

$$e_3 := 1.25 + 0.75\cdot\chi_w^{2} = 2.33$$

The verification formula, web, V factor according to Zizza.

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots} \leq 1 = 1$$
$$+ -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots} = 0.981$$
$$+ -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}$$

Stiffeners

Reduction factors, cross sectional parts with free edge

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^{2}}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 eq 4.3 Cross sectional parts with free edge

$$\psi := 1$$

$$\rho_{px.st.1} := \rho_{4.3}(\lambda_p, \psi) = 1$$

Column buckling, $\chi_{cx.fl}$:

$$\alpha_{unstiff} = 0.21$$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$$\Phi_{c.x.st.1} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot(\lambda_p - 0.2) + \lambda_p^{2}\right] = 0.777$$ Use λ_p, EC3-1-5 Ch. 10

$$\chi_{cx.st.1} := \frac{1}{\Phi_{c.x.st.1} + \sqrt{\Phi_{c.x.st.1}^{2} - \lambda_p^{2}}} = 0.859$$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$$\xi_{x.st.1} := \max\left(\min\left(\frac{\sigma_{cr.p.st.1}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$$

$$\rho_{cx.st.1} := (\rho_{px.st.1} - \chi_{cx.st.1})\cdot\xi_{x.st.1}\cdot(2 - \xi_{x.st.1}) + \chi_{cx.st.1} = 1$$ EC1993-1-5 eq.4.13

$$\xi_{x.st.2} := \max\left(\min\left(\frac{\sigma_{cr.p.st.2}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$$

The verification formula, stiffeners, with σ_{xEd} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \leq 1 = 1 \qquad \left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 = 0.202$$

Flange

Reduction factor, cross sectional parts with free edge

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^{2}}, 1\right) & \text{otherwise} \end{cases}$$

SS-en 1993-1-5(eq4.3) cross sectional parts with free edge

$$\psi := 1$$

$$\rho_{px.fl} := \rho_{4.3}(\lambda_p, \psi) = 1$$

Column buckling, $\chi_{cx.fl}$:

$$\alpha_{unstiff} = 0.21$$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$$\Phi_{c.x.fl} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot(\lambda_p - 0.2) + \lambda_p^{2}\right] = 0.777$$ Use λ_p, see EC3-1-5 Ch. 10

$$\chi_{cx.fl} := \frac{1}{\Phi_{c.x.fl} + \sqrt{\Phi_{c.x.fl}^{2} - \lambda_p^{2}}} = 0.859$$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$$\xi_{x.fl} := \max\left(\min\left(\frac{\sigma_{cr.p.fl}}{\sigma_{cr.c.fl}} - 1, 1\right), 0\right) = 1$$ Platelike behaviour

$\rho_{cx.fl} := \left(\rho_{px.fl} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_{x.fl}\right) + \chi_{cx.fl} = 1$ EC1993-1-5 eq.4.13

$\xi_3 := \max\left(1.25 + 0.75\cdot\chi_w^{\,2}, 2\right) = 2.33$

The verification formula, flange:

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 \le 1 = 1$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 = 0.995$$

The verification formula, flange, V factor according to Zizza.

$\psi_{tp} := 1$ Uniform compression

$c_{fl} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

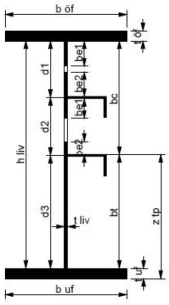
$\rho_{fl.x.V} := \min\left[c_{fl}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.999$

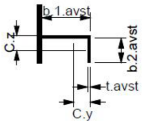
$\rho_{c.fl.x.V} := \left(\rho_{fl.x.V} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_x\right) + \chi_{cx.fl} = 0.999$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} \le 1 = 1$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.tp}}{\left(\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}\right)}\right]^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} = 0.998$$

Calculation of a longitudinal stiffened I-beam with the RSM - 350mm horizontal stiffener





Material parameters:

$\gamma_M := 1$

$E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$f_{yk} := \begin{pmatrix} f_y(16mm) \\ f_y(40mm) \\ f_y(41mm) \end{pmatrix} = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Area

$A_{öf} := t_{öf} \cdot b_{öf}$ $A_{liv} := t_{web} \cdot h_{liv}$ $A_{uf} := t_{uf} \cdot b_{uf}$

$A_{avst} := t_{avst} \cdot (b_{1.avst} + b_{2.avst})$ $A_{tot} := A_{öf} + A_{liv} + A_{uf} + 2 \cdot A_{avst} = 6.256 \times 10^4 \cdot mm^2$

Change of centre of gravity in y- and z-direction

$$C_y := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{b_{1.avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right)}{A_{avst}} = 140.432 \cdot mm$$

$$C_z := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(t_{avst} + \frac{b_{2.avst}}{2}\right)}{A_{avst}} = 16.432 \cdot mm$$

Geometry

$t_{öf} := 25mm$

$b_{öf} := 560mm$

$t_{web} := 12mm$

$h_{liv} := 2m$

$d_1 := 0.5m$

$d_2 := 0.5m$

$d_3 := 1m$

$t_{uf} := 25mm$

$b_{uf} := 560mm$

Horizontal stiffeners:

$b_{1.avst} := 350mm$

$b_{2.avst} := 90mm$

$t_{avst} := 12mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

Number of longitudinal stiffeners:

$n := 2$

Poissons ratio

$\nu := 0.3$

Distance from bottom fibre to centre of gravity

$$z_{tp} := \frac{\begin{aligned} &A_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + A_{liv} \cdot \left(t_{uf} + \frac{h_{liv}}{2}\right) + A_{öf} \cdot \left(t_{uf} + h_{liv} + \frac{t_{öf}}{2}\right) \dots \\ &+ A_{avst} \cdot (t_{uf} + d_2 + d_3 - C_z) + A_{avst} \cdot (t_{uf} + d_3 - C_z) \end{aligned}}{A_{tot}} = 1.064\,m$$

Moment of inirtia, stiffeners $I_{z.avst}$

$$I_{z.avst} := \frac{b_{2.avst} \cdot t_{avst}^3}{12} + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2} - C_y\right)^2 \dots + \frac{t_{avst} \cdot b_{1.avst}^3}{12} + t_{avst} \cdot b_{1.avst} \cdot \left(\frac{b_{1.avst}}{2} - C_y\right)^2 = 6.742 \times 10^{-5}\,m^4$$

The stiffeners area and moment of inertia fig A.1

$b_1 := d_1$

$b_{3.c} := t_{uf} + d_3 + d_2 - z_{tp} = 0.461\,m$

Highest stress(under the flange) is set to the yield stress

$$\sigma_{max} := f_y = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Stress at a z-koord, bottom flange in tenstion

$$\sigma_{neg}(z) := \left(\frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}\right) \cdot -1$$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$$

$\psi_{liv.1} := \psi_{neg}(t_{uf} + d_2 + d_3, t_{uf} + h_{liv}) = 0.443$

$\psi_{liv.2} := \psi_{neg}(t_{uf} + d_3, t_{uf} + d_2 + d_3) = -0.086$

$\psi_1 := \psi_{liv.1} = 0.443$

$\psi_3 := \psi_{liv.2} = -0.086$

$b_{1.inf} := \frac{3 - \psi_1}{5 - \psi_1} \cdot b_1 = 280.577 \cdot mm$

$b_{3.sup} := 0.4 \cdot b_{3.c} = 184.23 \cdot mm$

$A_{sl.1} := A_{avst} + t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) = 0.011\,m^2$

$y_{tp.sl.1} := \frac{t_{web} \cdot (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot \frac{t_{web}}{2} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web})}{A_{sl.1}} = 109.457 \cdot mm$

Stiffeners moment of inertia out from the web plate

$I_{z.sl.1} := I_{z.avst} + A_{avst} \cdot (b_{1.avst} - C_y + t_{web} - y_{tp.sl.1})^2 \ldots + \frac{(b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web}^3}{12} \ldots + (b_{1.inf} + b_{3.sup} + t_{avst}) \cdot t_{web} \cdot \left(\frac{t_{web}}{2} - y_{tp.sl.1}\right)^2 = 1.951 \times 10^{-4}\,m^4$

Moment of inertia, Von mises stress and alpha critical

$q_d := 156\,\frac{kN}{m}$ $I_y := 0.0379 m^4$ From EWM Ramboll

$\sigma_{v.unit} := \frac{2.31MPa}{\frac{kN}{m}}$ From FEM, close to middle of the span in top flange

$\sigma_V := \sigma_{v.unit} \cdot q_d = 360.36 \cdot MPa$

$\alpha_{cr.UNIT} := 342.6 \cdot \frac{kN}{m}$ Eigenvalue from FEM SOFiSTiK

$\alpha_{cr_fem} := \frac{\alpha_{cr.UNIT}}{q_d} = 2.196$

Load global

$V_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)}{2} = 2.086 \times 10^3 \cdot kN$

$M_{Ed} := \frac{q_d \cdot (s_{ts} \cdot 3)^2}{8} = 1.395 \times 10^4 \cdot kN \cdot m$

$V_{Ed.mid} := 0$

Web

$W_{y.web} := \frac{I_y}{h_{liv} + t_{uf} + t_{\ddot{o}f} - z_{tp}} = 0.038 \cdot m^3$

$\sigma_{x.Ed.web} := \frac{M_{Ed}}{W_{y.web}} = 362.637 \cdot MPa$

$\sigma_{z.Ed.web} := \frac{q_d}{t_{web}} = 13 \cdot MPa$

$S := t_{\ddot{o}f} \cdot b_{\ddot{o}f} \cdot \left(t_{uf} + h_{liv} + \frac{t_{\ddot{o}f}}{2} - z_{tp}\right) + \frac{h_{liv}}{2} \cdot t_{web} \cdot \left(t_{uf} + \frac{h_{liv}}{2} + \frac{h_{liv}}{4} - z_{tp}\right) = 0.019 \cdot m^3$

$\tau_{Ed.web} := \frac{V_{Ed} \cdot S}{I_y \cdot t_{web}} = 87.828 \cdot MPa$

$\tau_{Ed.web.2mfromsupport} := \frac{(V_{Ed} - q_d \cdot 2m) \cdot S}{I_y \cdot t_{web}} = 74.691 \cdot MPa$

Calculation of .von mises stess analyttical

$\sigma_{v.analytical} := \sqrt{\sigma_{x.Ed.web}^2 + \sigma_{z.Ed.web}^2 - \sigma_{x.Ed.web} \cdot \sigma_{z.Ed.web} + 3 \cdot \tau_{Ed.web}^2} = 387.43 \cdot MPa$

Stiffeners

Stiffener over

$W_{y.st.o} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1} = 0.082 \cdot m^3$

$\sigma_{x.Ed.st.o} := \frac{M_{Ed}}{W_{y.st.o}} = 169.466 \cdot MPa$

Stiffener under

$W_{y.st.u} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1 - d_2} = -0.961 \cdot m^3$

$\sigma_{x.Ed.st.u} := \frac{M_{Ed}}{W_{y.st.u}} = -14.507 \cdot MPa$

Flange

$W_{y.fl} := \frac{I_y}{\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}} = 0.039 \cdot m^3$

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{W_{y.fl}} = 358.038 \cdot MPa$

$S_{tp} := \frac{t_{\ddot{o}f}}{2} \cdot b_{\ddot{o}f} \cdot \left(\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}\right) = 6.812 \times 10^{-3} \cdot m^3$

$\tau_{Ed.tp.mid} := \frac{V_{Ed.mid} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0 \cdot MPa$

$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_y \cdot b_{\ddot{o}f}} = 0.669 \cdot MPa$

Calculation of load amplifiers, $\alpha_{ult,k}$, with von mises stress taken from sofistik.

-Formula for von mises in the denominator in $\alpha_{ult.k}$

$\alpha_{ult.k_mises} := \frac{f_{y2}}{\sigma_V} = 0.999$ σ_V from sofistik EC3-1-5 Ch.10(4) eq.10.3

Global slenderness $\lambda_{p,mises}$

$\lambda_p := \sqrt{\frac{\alpha_{ult.k_mises}}{\alpha_{cr_fem}}} = 0.674$

Flange

Flange, calculation of $\sigma_{cr,p}$:

$\psi_{flange} := 1$ $k_{\sigma.flange} := 0.43$ Cross sectional parts in compression with free edge SS-EN 1993-1-5 table 4.2

$\sigma_{E.flange} := \frac{\pi^2 \cdot E \cdot t_{uf}^2}{12 \cdot (1 - \nu^2) \cdot \left(\frac{b_{uf} - t_{web}}{2}\right)^2} = 1.58 \times 10^3 \cdot MPa$

$\sigma_{cr.p.fl} := k_{\sigma.flange} \cdot \sigma_{E.flange} = 679.428 \cdot MPa$

Flange; calculation of elastic critical column buckling stress $\sigma_{cr,c}$:

$\sigma_{cr.c.fl} := \frac{\pi^2 \cdot E \cdot t_{\ddot{o}f}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 1.493 \cdot MPa$ Unstiffened plate

Avstyvare; beräkning av $\sigma_{cr,p}$:

$\psi_{st} := 1$

$k_{\sigma.st} := 0.43$

$$\sigma_{E.st.1} := \frac{\pi^2 \cdot E \cdot t_{avst}^{\ 2}}{12 \cdot \left(1 - \nu^2\right) \cdot \left(b_{1.avst}\right)^2} = 223.112 \cdot MPa$$ Part 1 stiffener

$$\sigma_{E.st.2} := \frac{\pi^2 \cdot E \cdot t_{avst}^{\ 2}}{12 \cdot \left(1 - \nu^2\right) \cdot \left(b_{2.avst}\right)^2} = 3.374 \times 10^3 \cdot MPa$$ Part 2 stiffener

$\sigma_{cr.p.st.1} := k_{\sigma.st} \cdot \sigma_{E.st.1} = 95.938 \cdot MPa$

$\sigma_{cr.p.st.2} := k_{\sigma.st} \cdot \sigma_{E.st.2} = 1.451 \times 10^3 \cdot MPa$

Stiffener; calculation of elastic critical column buckling stress $\sigma_{cr.c}$:

$$\sigma_{cr.c.st} := \frac{\pi^2 \cdot E \cdot t_{avst}^{\ 2}}{12 \cdot \left(1 - \nu^2\right) \cdot s_{ts}^{\ 2}} = 0.344 \cdot MPa$$ Unstiffened plate

Calculation of the reduction factor χ_w for shear stess:

$\eta := 1.2$ EC3-1-5 Ch.5.1

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{0.7 + \lambda_p} & \text{otherwise} \end{cases}$$ EC3-1-5 tab.5.1

$\chi_w = 1.2$

WEB

Web; calculation of χ_{cx} (that depends on λ_p)

$\alpha := 0.49$

$e_2 := y_{tp.sl.1} - \frac{t_{web}}{2} = 103.457 \cdot mm$

$e_1 := C_y - e_2 - \frac{t_{web}}{2} = 30.975 \cdot mm$

$e := \max(e_1, e_2)$ EC3-1-5 4.5.3(5)

$i := \sqrt{\frac{I_{z.sl.1}}{A_{sl.1}}} = 0.133\,m$ EC3-1-5 4.5.3(5)

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.56$ EC3-1-5 4.5.3(5), stiffened plates

$\Phi_c := 0.5 \cdot \left[1 + \alpha_e \cdot \left(\lambda_p - 0.2\right) + \lambda_p^{\ 2}\right] = 0.86$

$$\chi_{cx} := \min\left(\frac{1}{\Phi_c + \sqrt{\Phi_c^{\ 2} - \lambda_p^{\ 2}}}, 1\right) = 0.717$$ EC3-1-1 eq. 6.49

Web; Calculation reduction factors ρ_p

Internal

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max[(3 + \psi), 0]}{\lambda_p^{\ 2}}, 1\right] & \text{otherwise} \end{cases}$$ SS-EN 1993-1-5(eq4.2) Internal cross sectional parts

$\psi_{liv} := \psi_{neg}(t_{uf}, t_{uf} + h_{liv}) = -1.082$ Stress distribution, $\psi_x = \sigma_2/\sigma_1$

$\rho_{px} := \rho_{4.2}(\lambda_p, \psi_{liv}) = 1$

Calculation of amplifers, $\sigma_{cr.p}$ and $\sigma_{cr.c}$, for column like buckling in x direction for the plate

$b_2 := d_2 \qquad b := b_1 + b_2$

$$a_c := 4.33 \cdot \sqrt[4]{\frac{I_{z.sl.1} \cdot b_1^{\ 2} \cdot b_2^{\ 2}}{t_{web}^{\ 3} \cdot b}} = 7.057\,m$$

$$\sigma_{cr.sl.p} := \begin{cases} \left(\frac{1.05 \cdot E}{A_{sl.1}} \cdot \sqrt{\frac{I_{z.sl.1} \cdot t_{web}^{\ 3} \cdot b}{b_1 \cdot b_2}}\right) & \text{if } s_{ts} \ge a_c \\ \left[\frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^{\ 2}} + \frac{E \cdot t_{web}^{\ 3} \cdot b \cdot s_{ts}^{\ 2}}{4 \cdot \pi^2 \cdot \left(1 - \nu^2\right) \cdot A_{sl.1} \cdot b_1^{\ 2} \cdot b_2^{\ 2}}\right] & \text{if } s_{ts} \le a_c \end{cases}$$ Eq. A.4

$b_c := t_{uf} + h_{liv} - z_{tp} = 0.961\,m$

$b_{sl.1} := b_c - d_1 = 0.461\,m$

$b_t := h_{liv} - b_c = 1.039\,m$

$$\sigma_{cr.p} := \sigma_{cr.sl.p} \cdot \frac{b_c}{b_{sl.1}} = 3.07 \times 10^3 \cdot MPa$$ Stiffened plate with one longitudinal stiffener in the compression zone. EC3-1-5 Ch. A.2.2

$$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^{\ 2}} = 462.56 \cdot MPa$$ Stiffened plate. EC3-1-5 Ch. 4.5.3

$\sigma_{cr.c} := \sigma_{cr.sl} \cdot \frac{b_c}{b_{sl.1}} = 964.716 \cdot MPa$

Interaction plate like/column like

$\xi_x := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.web} := \left(\rho_{px} - \chi_{cx}\right) \cdot \xi_x \cdot \left(2 - \xi_x\right) + \chi_{cx} = 1$

$$\rho_{cx.web} := \begin{cases} \left(\rho_{px} - \chi_{cx}\right) \cdot \xi_x \cdot \left(2 - \xi_x\right) + \chi_{cx} & \text{if } \frac{\left[\left(\rho_{px} - \chi_{cx}\right) \cdot \xi_x \cdot \left(2 - \xi_x\right) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px} - \chi_{cx}\right) \cdot \xi_x \cdot \left(2 - \xi_x\right) + \chi_{cx}\right] \cdot \alpha_{ult.k_mises}}{\gamma_M} \ge 1 \end{cases} = 1$$ According to EC1993-1-5 eq.10.1

Transverse stresses (z-direction), calculation of ρ_{pz}:

$\lambda_{p0} := 0.8 \qquad \alpha_p := 0.34$ Table B.1

$\varphi_p := 0.5 \cdot \left[1 + \alpha_p \cdot \left(\lambda_p - \lambda_{p0}\right) + \lambda_p\right] = 0.816$

$$\rho_{p.z} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\varphi_p + \sqrt{\varphi_p^{\ 2} - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.21 - 0.139i$$ EC1993-1-5, eq. B.1

Elastic critical buckling stress and the buckling stress for a planet that is not stiffened, seen from the z-direction:

$$\sigma_{cr.c.z} := \frac{\pi^2 \cdot E \cdot t_{web}^{\ 2}}{12 \cdot \left(1 - \nu^2\right) \cdot h_{liv}^{\ 2}} = 6.833 \cdot MPa$$

$$\sigma_{E.z} := \frac{\pi^2 \cdot E \cdot t_{web}^{\ 2}}{12 \cdot \left(1 - \nu^2\right) \cdot s_{ts}^{\ 2}} = 0.344 \cdot MPa$$

$k_{\sigma z} := 4$

$\sigma_{cr.p.z} := k_{\sigma z} \cdot \sigma_{E.z} = 1.376 \cdot MPa$

Column buckling, χ_{cz}:

$\alpha_{unstiff} := 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.z} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot(\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.777$ Use λ_p according to EC3-1-5 Chapter 10

$\chi_{cz} := \dfrac{1}{\Phi_{c.z} + \sqrt{\Phi_{c.z}^{\,2} - \lambda_p^{\,2}}} = 0.859$ EC3-1-1 eq. 6.49

Interaction plate like/column like:

$\xi_z := \max\left(\min\left(\dfrac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 1\right), 0\right) = 0$

$\rho_{cz.web} := (\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz} = 0.859$

$$\rho_{cz.web} := \begin{cases} (\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz} & \text{if } \dfrac{\left[(\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) \ldots\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} + \chi_{cz} < 1 \\ 1 & \text{if } \dfrac{\left[(\rho_{p.z} - \chi_{cz})\cdot\xi_z\cdot(2 - \xi_z) + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} \geq 1 \end{cases} = 0.85$$

According to EC1993-1-5 eq.10.1

The verification formula, web, with σ_{xEd}, σ_{zEd} and τ_{Ed} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots \leq 1 = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \ldots = 0.986$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

Verification with V-factor according to Zizza

$c_{web} := \min\left(1.25 - 0.25\cdot\psi_{Iv}, 1.25\right) = 1.25$

$\rho_{web.x.V} := \min\left[c_{web}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 1$

$\rho_{c.web.x.V} := (\rho_{web.x.V} - \chi_{cx})\cdot\xi_x\cdot(2 - \xi_x) + \chi_{cx} = 1$

$\xi_{z.z} := \max\left(\dfrac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 0\right) = 0$

$f := \lambda_p^{\,-0.5\cdot\ln\left(\xi_{z.z} + 1\right)^{0.9}} = 0$

$\rho_{c.web.z.V} := \chi_{cz} + (\rho_{p.z} - \chi_{cz})\cdot f = 0.859$

$V_{web} := \rho_{c.web.x.V}\cdot\rho_{c.web.z.V} = 0.859$

$e_3 := 1.25 + 0.75\cdot\chi_w^{\,2} = 2.33$

The verification formula, web, V factor according to Zizza.

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} \leq 1 = 1$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \ldots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} = 0.978$$

Stiffeners

Reduction factors, cross sectional parts with free edge

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$$

SS-en 1993-1-5(eq4.3) cross sections with free edge

$\psi := 1$

$\rho_{px.st.1} := \rho_{4.3}(\lambda_p, \psi) = 1$

Column buckling, $\chi_{cx.flt}$:

$\alpha_{unstiff} = 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.x.st.1} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot(\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.777$ According tp EC3-1-5 Chapter 10, use λ_p

$\chi_{cx.st.1} := \dfrac{1}{\Phi_{c.x.st.1} + \sqrt{\Phi_{c.x.st.1}^{\,2} - \lambda_p^{\,2}}} = 0.859$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$\xi_{x.st.1} := \max\left(\min\left(\dfrac{\sigma_{cr.p.st.1}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.st.1} := (\rho_{px.st.1} - \chi_{cx.st.1})\cdot\xi_{x.st.1}\cdot(2 - \xi_{x.st.1}) + \chi_{cx.st.1} = 1$ EC1993-1-5 eq.4.13

$\xi_{x.st.2} := \max\left(\min\left(\dfrac{\sigma_{cr.p.st.2}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$

The verification formula, stiffeners, with σ_{xEd} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \leq 1 = 1 \qquad \left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 = 0.199$$

Flange

Reduction factor, cross sectional parts with free edge

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$$

SS-en 1993-1-5(eq4.3) cross sectional parts with free edge

$\psi := 1$

$\rho_{px.fl} := \rho_{4.3}(\lambda_p, \psi) = 1$

Column buckling, $\chi_{cx.flt}$:

$\alpha_{unstiff} = 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.x.fl} := 0.5 \cdot \left[1 + \alpha_{unstiff} \cdot \left(\lambda_p - 0.2\right) + \lambda_p^{\,2}\right] = 0.777$ Use λ_p, see EC3-1-5 Ch. 10

$\chi_{cx.fl} := \frac{1}{\Phi_{c.x.fl} + \sqrt{\Phi_{c.x.fl}^{\,2} - \lambda_p^{\,2}}} = 0.859$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$\xi_{x.fl} := \max\left(\min\left(\frac{\sigma_{cr.p.fl}}{\sigma_{cr.c.fl}} - 1, 1\right), 0\right) = 1$ Platelike behaviour

$\rho_{cx.fl} := \left(\rho_{px.fl} - \chi_{cx.fl}\right) \cdot \xi_{x.fl} \cdot \left(2 - \xi_{x.fl}\right) + \chi_{cx.fl} = 1$ EC1993-1-5 eq.4.13

$e_3 := \max\left(1.25 + 0.75 \cdot \chi_w^{\,2}, 2\right) = 2.33$

The verification formula, flange:

$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl} \cdot f_{y_2}}{\gamma_M}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w \cdot f_{y_2}}{\gamma_M}}\right)^2 \le 1 = 1$

$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl} \cdot f_{y_2}}{\gamma_M}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w \cdot f_{y_2}}{\gamma_M}}\right)^2 = 0.989$

The verification formula, flange, V factor according to Zizza.

$\psi_{tp} := 1$ Uniform compression

$c_{fl} := \min\left(1.25 - 0.25 \cdot \psi_{tp}, 1.25\right) = 1$

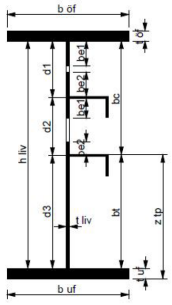
$\rho_{fl.x.V} := \min\left[c_{fl} \cdot \left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.999$

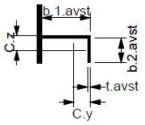
$\rho_{c.fl.x.V} := \left(\rho_{fl.x.V} - \chi_{cx.fl}\right) \cdot \xi_{x.fl} \cdot \left(2 - \xi_x\right) + \chi_{cx.fl} = 0.999$

$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.fl.x.V} \cdot f_{y_2}}{\gamma_M}}\right)^2 + \left(\frac{\sqrt{3} \cdot \tau_{Ed.tp}}{\frac{\chi_w \cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} \le 1 = 1$

$\sqrt{\left[\frac{\sigma_{x.Ed.tp}}{\left(\frac{\rho_{c.fl.x.V} \cdot f_{y_2}}{\gamma_M}\right)}\right]^2 + \left(\frac{\sqrt{3} \cdot \tau_{Ed.tp}}{\frac{\chi_w \cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} = 0.995$

Calculation of a longitudinal stiffened I-beam with the RSM - 400mm horizontal stiffener





Geometry

$t_{öf} := 25mm$

$b_{öf} := 560mm$

$t_{web} := 12mm$

$h_{liv} := 2m$

$d_1 := 0.5m$

$d_2 := 0.5m$

$d_3 := 1m$

$t_{uf} := 25mm$

$b_{uf} := 560mm$

Horizontal stiffeners:

$b_{1.avst} := 400mm$

$b_{2.avst} := 90mm$

$t_{avst} := 12mm$

Distance between vertical stiffeners:

$s_{ts} := 8.914m$

Number of longitudinal stiffeners:

$n := 2$

Poissons ratio

$\nu := 0.3$

Material parameters:

$\gamma_M := 1$

$E := 210GPa$

$$f_y(t) := \begin{cases} \frac{390MPa}{\gamma_M} & \text{if } t \le 16mm \\ \frac{380MPa}{\gamma_M} & \text{if } 16mm < t \le 40mm \\ \frac{360MPa}{\gamma_M} & \text{otherwise} \end{cases}$$

$$f_{yk} := \begin{pmatrix} f_y(16mm) \\ f_y(40mm) \\ f_y(41mm) \end{pmatrix} = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Area

$A_{öf} := t_{öf} \cdot b_{öf}$ $A_{liv} := t_{web} \cdot h_{liv}$ $A_{uf} := t_{uf} \cdot b_{uf}$

$A_{avst} := t_{avst} \cdot (b_{1.avst} + b_{2.avst})$ $A_{tot} := A_{öf} + A_{liv} + A_{uf} + 2 \cdot A_{avst} = 6.376 \times 10^{4} \cdot mm^2$

Change of centre of gravity in y- and z-direction

$$C_y := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{b_{1.avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right)}{A_{avst}} = 164.367 \cdot mm$$

$$C_z := \frac{b_{1.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2}\right) + b_{2.avst} \cdot t_{avst} \cdot \left(t_{avst} + \frac{b_{2.avst}}{2}\right)}{A_{avst}} = 15.367 \cdot mm$$

Distance from bottom fibre to centre of gravity

$$z_{tp} := \frac{\begin{matrix} A_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + A_{liv} \cdot \left(t_{uf} + \frac{h_{liv}}{2}\right) + A_{öf} \cdot \left(t_{uf} + h_{liv} + \frac{t_{öf}}{2}\right) \dots \\ + A_{avst} \cdot (t_{uf} + d_2 + d_3 - C_z) + A_{avst} \cdot (t_{uf} + d_3 - C_z) \end{matrix}}{A_{tot}} = 1.068\,m$$

Moment of inirtia, stiffeners $I_{z.avst}$

$$I_{z.avst} := \frac{b_{2.avst} \cdot t_{avst}^3}{12} + b_{2.avst} \cdot t_{avst} \cdot \left(\frac{t_{avst}}{2} - C_y\right)^2 \dots + \frac{t_{avst} \cdot b_{1.avst}^3}{12} + t_{avst} \cdot b_{1.avst} \cdot \left(\frac{b_{1.avst}}{2} - C_y\right)^2 = 9.719 \times 10^{-5}\,m^4$$

The stiffeners area and moment of inertia fig A.1

$b_1 := d_1$

$b_{3.c} := t_{uf} + d_3 + d_2 - z_{tp} = 0.457\,m$

Highest stress(under the flange) is set to the yield stress

$$\sigma_{max} := f_y = \begin{pmatrix} 390 \\ 380 \\ 360 \end{pmatrix} \cdot MPa$$

Stress at a z-koord, bottom flange in tenstion

$$\sigma_{neg}(z) := \left(\frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}\right) \cdot -1$$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$$

$\psi_{liv.1} := \psi_{neg}(t_{uf} + d_2 + d_3, t_{uf} + h_{liv}) = 0.441$

$\psi_{liv.2} := \psi_{neg}(t_{uf} + d_3, t_{uf} + d_2 + d_3) = -0.095$

$\psi_1 := \psi_{liv.1} = 0.441$

$\psi_3 := \psi_{liv.2} = -0.095$

$$b_{1.inf} := \frac{3-\psi_1}{5-\psi_1}\cdot b_1 = 280.67\cdot mm$$

$$b_{3.sup} := 0.4\cdot b_{3.c} = 182.69\cdot mm$$

$$A_{sl.1} := A_{avst} + t_{web}\cdot\left(b_{1.inf} + b_{3.sup} + t_{avst}\right) = 0.012\,m^2$$

$$y_{tp.sl.1} := \frac{t_{web}\cdot\left(b_{1.inf} + b_{3.sup} + t_{avst}\right)\cdot\frac{t_{web}}{2} + A_{avst}\cdot\left(b_{1.avst} - C_y + t_{web}\right)}{A_{sl.1}} = 128.649\cdot mm$$

Stiffeners moment of inertia out from the web plate

$$I_{z.sl.1} := I_{z.avst} + A_{avst}\cdot\left(b_{1.avst} - C_y + t_{web} - y_{tp.sl.1}\right)^2 \ldots + \frac{\left(b_{1.inf} + b_{3.sup} + t_{avst}\right)\cdot t_{web}^3}{12} \ldots + \left(b_{1.inf} + b_{3.sup} + t_{avst}\right)\cdot t_{web}\cdot\left(\frac{t_{web}}{2} - y_{tp.sl.1}\right)^2 = 2.663\times 10^{-4}\,m^4$$

Moment of inertia, Von mises stress and alpha critical

$q_d := 157\frac{kN}{m}$ $\quad I_y := 0.038m^4$ From EWM Ramboll

$$\sigma_{v.unit} := \frac{2.29MPa}{\frac{kN}{m}}$$

From FEM, somewhere close to middle of the span in top flange

$$\sigma_V := \sigma_{v.unit}\cdot q_d = 359.53\cdot MPa$$

$$\alpha_{cr.UNIT} := 345.96\cdot\frac{kN}{m}$$

Eigenvalue from FEM SOFiSTiK

$$\alpha_{cr_fem} := \frac{\alpha_{cr.UNIT}}{q_d} = 2.204$$

Load global

$$V_{Ed} := \frac{q_d\cdot\left(s_{ts}\cdot 3\right)}{2} = 2.099\times 10^3\cdot kN$$

$$M_{Ed} := \frac{q_d\cdot\left(s_{ts}\cdot 3\right)^2}{8} = 1.403\times 10^4\cdot kN\cdot m$$

$$V_{Ed.mid} := 0$$

Web

$$W_{y.web} := \frac{I_y}{h_{liv} + t_{uf} + t_{\ddot{o}f} - z_{tp}} = 0.039\cdot m^3$$

$$\sigma_{x.Ed.web} := \frac{M_{Ed}}{W_{y.web}} = 362.579\cdot MPa$$

$$\sigma_{z.Ed.web} := \frac{q_d}{t_{web}} = 13.083\cdot MPa$$

$$S := t_{\ddot{o}f}\cdot b_{\ddot{o}f}\cdot\left(t_{uf} + h_{liv} + \frac{t_{\ddot{o}f}}{2} - z_{tp}\right) + \frac{h_{liv}}{2}\cdot t_{web}\cdot\left(t_{uf} + \frac{h_{liv}}{2} + \frac{h_{liv}}{4} - z_{tp}\right) = 0.019\cdot m^3$$

$$\tau_{Ed.web} := \frac{V_{Ed}\cdot S}{I_y\cdot t_{web}} = 87.698\cdot MPa$$

$$\tau_{Ed.web.2mfromsupport} := \frac{\left(V_{Ed} - q_d\cdot 2m\right)\cdot S}{I_y\cdot t_{web}} = 74.58\cdot MPa$$

Calculation of .von mises stess analyttical

$$\sigma_{v.analytical} := \sqrt{\sigma_{x.Ed.web}^2 + \sigma_{z.Ed.web}^2 - \sigma_{x.Ed.web}\cdot\sigma_{z.Ed.web} + 3\cdot\tau_{Ed.web}^2} = 387.252\cdot MPa$$

Stiffeners

Stiffener over

$$W_{y.st.o} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1} = 0.083\cdot m^3$$

$$\sigma_{x.Ed.st.o} := \frac{M_{Ed}}{W_{y.st.o}} = 168.682\cdot MPa$$

Stiffener under

$$W_{y.st.u} := \frac{I_y}{h_{liv} + t_{uf} - z_{tp} - d_1 - d_2} = -0.878\cdot m^3$$

$$\sigma_{x.Ed.st.u} := \frac{M_{Ed}}{W_{y.st.u}} = -15.983\cdot MPa$$

Flange

$$W_{y.fl} := \frac{I_y}{\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}} = 0.039\cdot m^3$$

$$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{W_{y.fl}} = 357.963\cdot MPa$$

$$S_{tp} := \frac{t_{\ddot{o}f}}{2}\cdot b_{\ddot{o}f}\cdot\left(\frac{t_{\ddot{o}f}}{2} + t_{uf} + h_{liv} - z_{tp}\right) = 6.785\times 10^{-3}\cdot m^3$$

$$\tau_{Ed.tp.mid} := \frac{V_{Ed.mid}\cdot S_{tp}}{I_y\cdot b_{\ddot{o}f}} = 0\cdot MPa$$

$$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_y\cdot b_{\ddot{o}f}} = 0.669\cdot MPa$$

Calculation of load amplifiers, $\alpha_{ult,k}$, with von mises stress taken from sofistik.

-Formula for von mises in the denominator in α_{ultk}

$$\alpha_{ult.k_mises} := \frac{f_{y2}}{\sigma_V} = 1.001$$

σ_V from sofistik

EC3-1-5 Ch.10(4) eq.10.3

Global slenderness $\lambda_{p,mises}$

$$\lambda_p := \sqrt{\frac{\alpha_{ult.k_mises}}{\alpha_{cr_fem}}} = 0.674$$

Flange

Flange, calculation of $\sigma_{cr,p}$:

$\psi_{flange} := 1$ $\quad k_{\sigma.flange} := 0.43$

Cross sectional parts in compression with free edge SS-EN 1993-1-5 table 4.2

$$\sigma_{E.flange} := \frac{\pi^2\cdot E\cdot t_{uf}^2}{12\cdot\left(1-\nu^2\right)\cdot\left(\frac{b_{uf} - t_{web}}{2}\right)^2} = 1.58\times 10^3\cdot MPa$$

$$\sigma_{cr.p.fl} := k_{\sigma.flange}\cdot\sigma_{E.flange} = 679.428\cdot MPa$$

Flange; calculation of elastic critical column buckling stress $\sigma_{cr,c}$:

$$\sigma_{cr.c.fl} := \frac{\pi^2\cdot E\cdot t_{\ddot{o}f}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 1.493\cdot MPa$$

Unstiffened plate

Stiffener; calculation of $\sigma_{cr,p}$:

$\psi_{st} := 1$

$k_{\sigma.st} := 0.43$

$$\sigma_{E.st.1} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1-\nu^2) \cdot (b_{1.avst})^2} = 170.82 \cdot MPa$$ Part 1 stiffener

$$\sigma_{E.st.2} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1-\nu^2) \cdot (b_{2.avst})^2} = 3.374 \times 10^3 \cdot MPa$$ Part 2 stiffener

$\sigma_{cr.p.st.1} := k_{\sigma.st} \cdot \sigma_{E.st.1} = 73.453 \cdot MPa$

$\sigma_{cr.p.st.2} := k_{\sigma.st} \cdot \sigma_{E.st.2} = 1.451 \times 10^3 \cdot MPa$

Stiffener; calculation of elastic critical column buckling stress $\sigma_{cr.c}$:

$$\sigma_{cr.c.st} := \frac{\pi^2 \cdot E \cdot t_{avst}^2}{12 \cdot (1-\nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$$ Unstiffened plate

Calculation of the reduction factor χ_w for shear stess:

$\eta := 1.2$ EC3-1-5 Ch.5.1

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{0.7 + \lambda_p} & \text{otherwise} \end{cases}$$ EC3-1-5 tab.5.1

$\chi_w = 1.2$

WEB

Web; calculation of χ_{cx} (that depends on λ_p)

$\alpha := 0.49$

$e_2 := y_{tp.sl.1} - \frac{t_{web}}{2} = 122.649 \cdot mm$

$e_1 := C_y - e_2 - \frac{t_{web}}{2} = 35.719 \cdot mm$

$\mathcal{S} := \max(e_1, e_2)$ EC3-1-5 4.5.3(5)

$i := \sqrt{\frac{I_{z.sl.1}}{A_{sl.1}}} = 0.152\,m$ EC3-1-5 4.5.3(5)

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.563$ EC3-1-5 4.5.3(5), för stiffened plates

$\Phi_c := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.861$

$\chi_{cx} := \min\left(\frac{1}{\Phi_c + \sqrt{\Phi_c^2 - \lambda_p^2}}, 1\right) = 0.717$ EC3-1-1 eq. 6.49

Web; Calculation reduction factors ρ_p

Internal

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055 \cdot \psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max[(3+\psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$ SS-EN 1993-1-5(eq4.2) Internal cross sectional parts

$\psi_{liv} := \psi_{neg}(t_{uf}, t_{uf} + h_{liv}) = -1.09$ Stress distribution, $\psi_x = \sigma_2/\sigma_1$

$\rho_{px} := \rho_{4.2}(\lambda_p, \psi_{liv}) = 1$

Calculation of amplifers, $\sigma_{cr.p}$ and $\sigma_{cr.c}$, for column like buckling in x direction for the plate

$b_2 := d_2$ $b := b_1 + b_2$

$$a_c := 4.33 \cdot \sqrt[4]{\frac{I_{z.sl.1} \cdot b_1^2 \cdot b_2^2}{t_{web}^3 \cdot b}} = 7.628\,m$$

$$\sigma_{cr.sl.p} := \begin{cases} \left(\frac{1.05 \cdot E}{A_{sl.1}} \cdot \sqrt{\frac{I_{z.sl.1} \cdot t_{web}^3 \cdot b}{b_1 \cdot b_2}}\right) & \text{if } s_{ts} \ge a_c \\ \left[\frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} + \frac{E \cdot t_{web}^3 \cdot b \cdot s_{ts}^2}{4 \cdot \pi^2 \cdot (1-\nu^2) \cdot A_{sl.1} \cdot b_1^2 \cdot b_2^2}\right] & \text{if } s_{ts} \le a_c \end{cases}$$ Eq. A.4

$b_c := t_{uf} + h_{liv} - z_{tp} = 0.957\,m$

$b_{sl.1} := b_c - d_1 = 0.457\,m$

$b_t := h_{liv} - b_c = 1.043\,m$

$\sigma_{cr.p} := \sigma_{cr.sl.p} \cdot \frac{b_c}{b_{sl.1}} = 3.421 \times 10^3 \cdot MPa$ Stiffened plate with one longitudinal stiffener in the compression zone. EC3-1-5 Ch. A.2.2

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{z.sl.1}}{A_{sl.1} \cdot s_{ts}^2} = 599.652 \cdot MPa$ Stiffened plate. EC3-1-5 Ch. 4.5.3

$\sigma_{cr.c} := \sigma_{cr.sl} \cdot \frac{b_c}{b_{sl.1}} = 1.256 \times 10^3 \cdot MPa$

Interaction plate like/column like

$\xi_x := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.c}} - 1, 1\right), 0\right) = 1$

$\rho_{cx.web} := (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} = 1$

$$\rho_{cx.web} := \begin{cases} (\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx} & \text{if } \frac{[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}] \cdot \alpha_{ult.k_mises}}{\gamma_M} < 1 \\ 1 & \text{if } \frac{[(\rho_{px} - \chi_{cx}) \cdot \xi_x \cdot (2 - \xi_x) + \chi_{cx}] \cdot \alpha_{ult.k_mises}}{\gamma_M} \ge 1 \end{cases} = 1$$ According to EC1993-1-5 eq.10.1

Transverse stresses (z-direction), calculation of ρ_{pz}:

$\lambda_{p0} := 0.8$ $\alpha_p := 0.34$ Table B.1

$\varphi_p := 0.5 \cdot \left[1 + \alpha_p \cdot (\lambda_p - \lambda_{p0}) + \lambda_p\right] = 0.816$

$$\rho_{p.z} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.21 - 0.139i$$ EC1993-1-5, eq. B.1

Elastic critical buckling stress and the buckling stress for a planet that is not stiffened, seen from the z-direction:

$$\sigma_{cr.c.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1-\nu^2) \cdot h_{liv}^2} = 6.833 \cdot MPa$$

$$\sigma_{E.z} := \frac{\pi^2 \cdot E \cdot t_{web}^2}{12 \cdot (1-\nu^2) \cdot s_{ts}^2} = 0.344 \cdot MPa$$

$k_{\sigma z} := 4$

$\sigma_{cr.p.z} := k_{\sigma z} \cdot \sigma_{E.z} = 1.376 \cdot MPa$

Column buckling, χ_{cz}:

$\alpha_{unstiff} := 0.21$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$\Phi_{c.z} := 0.5 \cdot \left[1 + \alpha_{unstiff} \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.777$ Use λ_p according to EC3-1-5 Chapter 10

$$\chi_{cz} := \frac{1}{\Phi_{c.z} + \sqrt{\Phi_{c.z}^{2} - \lambda_p^{2}}} = 0.86$$ EC3-1-1 eq. 6.49

Interaction plate like/column like:

$$\xi_z := \max\left(\min\left(\frac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 1\right), 0\right) = 0$$

$$\rho_{cz.web} := \left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz} = 0.86$$

$$\rho_{cz.web} := \begin{cases} \left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz} & \text{if } \frac{\left[\left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) \cdots + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} < 1 \quad = 0.86 \\ 1 & \text{if } \frac{\left[\left(\rho_{p.z} - \chi_{cz}\right)\cdot\xi_z\cdot\left(2 - \xi_z\right) + \chi_{cz}\right]\cdot\alpha_{ult.k_mises}}{\gamma_M} \geq 1 \end{cases}$$ According to EC1993-1-5 eq.10.1

The verification formula, web, with σ_{xEd}, σ_{zEd} and τ_{Ed} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \cdots \leq 1 = 1$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

$$\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 - \left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{cx.web}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{cz.web}\cdot f_{y_1}}{\gamma_M}\right)}\right] \cdots = 0.985$$
$$+ 3\cdot\left[\frac{\tau_{Ed.web}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^2$$

Verification with V-factor according to Zizza

$$c_{web} := \min\left(1.25 - 0.25\cdot\psi_{liv}, 1.25\right) = 1.25$$

$$\rho_{web.x.V} := \min\left[c_{web}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{2}}\right), 1\right] = 1$$

$$\rho_{c.web.x.V} := \left(\rho_{web.x.V} - \chi_{cx}\right)\cdot\xi_x\cdot\left(2 - \xi_x\right) + \chi_{cx} = 1$$

$$\xi_{z.z} := \max\left(\frac{\sigma_{cr.p.z}}{\sigma_{cr.c.z}} - 1, 0\right) = 0$$

$$f := \lambda_p^{-0.5}\cdot\ln\left(\xi_{z.z} + 1\right)^{0.9} = 0$$

$$\rho_{c.web.z.V} := \chi_{cz} + \left(\rho_{p.z} - \chi_{cz}\right)\cdot f = 0.86$$

$$V_{web} := \rho_{c.web.x.V}\cdot\rho_{c.web.z.V} = 0.86$$

$$e_3 := 1.25 + 0.75\cdot\chi_w^{2} = 2.33$$

The verification formula, web, V factor according to Zizza.

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \cdots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} \leq 1 = 1$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 + \left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \cdots + -V_{web}\cdot\left[\frac{\sigma_{x.Ed.web}}{\left(\frac{\rho_{c.web.x.V}\cdot f_{y_1}}{\gamma_M}\right)}\right]\cdot\left[\frac{\sigma_{z.Ed.web}}{\left(\frac{\rho_{c.web.z.V}\cdot f_{y_1}}{\gamma_M}\right)}\right] + \left[\frac{\tau_{Ed.web}\cdot\sqrt{3}}{\left(\frac{\chi_w\cdot f_{y_1}}{\gamma_M}\right)}\right]^{e_3}} = 0.978$$

Stiffeners

Reduction factors, cross sectional parts with free edge

$$\rho_{4.3}\left(\lambda_p, \psi\right) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^{2}}, 1\right) & \text{otherwise} \end{cases}$$ SS-EN 1993-1-5 eq 4.3 Cross sectional parts with free edge

$$\psi := 1$$

$$\rho_{px.st.1} := \rho_{4.3}\left(\lambda_p, \psi\right) = 1$$

Column buckling, $\chi_{cx.fl}$:

$$\alpha_{unstiff} = 0.21$$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$$\Phi_{c.x.st.1} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{2}\right] = 0.777$$ Use λ_p, EC3-1-5 Ch. 10

$$\chi_{cx.st.1} := \frac{1}{\Phi_{c.x.st.1} + \sqrt{\Phi_{c.x.st.1}^{2} - \lambda_p^{2}}} = 0.86$$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$$\xi_{x.st.1} := \max\left(\min\left(\frac{\sigma_{cr.p.st.1}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$$

$$\rho_{cx.st.1} := \left(\rho_{px.st.1} - \chi_{cx.st.1}\right)\cdot\xi_{x.st.1}\cdot\left(2 - \xi_{x.st.1}\right) + \chi_{cx.st.1} = 1$$ EC1993-1-5 eq.4.13

$$\xi_{x.st.2} := \max\left(\min\left(\frac{\sigma_{cr.p.st.2}}{\sigma_{cr.c.st}} - 1, 1\right), 0\right) = 1$$

The verification formula, stiffeners, with σ_{xEd} taken from sofistik:

$$\left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 \leq 1 = 1 \qquad \left[\frac{\sigma_{x.Ed.st.o}}{\left(\frac{\rho_{cx.st.1}\cdot f_{y_1}}{\gamma_M}\right)}\right]^2 = 0.197$$

Flange

Reduction factor, cross sectional parts with free edge

$$\rho_{4.3}\left(\lambda_p, \psi\right) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^{2}}, 1\right) & \text{otherwise} \end{cases}$$ SS-en 1993-1-5(eq4.3) cross sectional parts with free edge

$$\psi := 1$$

$$\rho_{px.fl} := \rho_{4.3}\left(\lambda_p, \psi\right) = 1$$

Column buckling, $\chi_{cx.fl}$:

$$\alpha_{unstiff} = 0.21$$ EC1993-1-5 Ch. 4.5.3(5), unstiffened plate

$$\Phi_{c.x.fl} := 0.5\cdot\left[1 + \alpha_{unstiff}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^{2}\right] = 0.777$$ Use λ_p, see EC3-1-5 Ch. 10

$$\chi_{cx.fl} := \frac{1}{\Phi_{c.x.fl} + \sqrt{\Phi_{c.x.fl}^{2} - \lambda_p^{2}}} = 0.86$$ EC3-1-1 eq. 6.49

Interaction plate like/column like

$$\xi_{x.fl} := \max\left(\min\left(\frac{\sigma_{cr.p.fl}}{\sigma_{cr.c.fl}} - 1, 1\right), 0\right) = 1$$ Platelike behaviour

$$\rho_{cx.fl} := \left(\rho_{px.fl} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_{x.fl}\right) + \chi_{cx.fl} = 1$$ EC1993-1-5 eq.4.13

$$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^{2}, 2\right) = 2.33$$

The verification formula, flange:

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 \leq 1 = 1$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{cx.fl}\cdot f_{y_2}}{\gamma_M}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp.mid}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^2 = 0.989$$

The verification formula, flange, V factor according to Zizza.

$\psi_{tp} := 1$ Uniform compression

$c_{fl} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{fl.x.V} := \min\left[c_{fl}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 0.999$

$\rho_{c.fl.x.V} := \left(\rho_{fl.x.V} - \chi_{cx.fl}\right)\cdot\xi_{x.fl}\cdot\left(2 - \xi_x\right) + \chi_{cx.fl} = 0.999$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} \le 1 = 1$$

$$\sqrt{\left[\frac{\sigma_{x.Ed.tp}}{\left(\frac{\rho_{c.fl.x.V}\cdot f_{y_2}}{\gamma_M}\right)}\right]^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_{y_2}}{\gamma_M}}\right)^{e_3}} = 0.995$$

C

Box girder

C.1 Original cross section

CII Original cross section EWM calculation
CIX Original cross section RSM calculation

EWM - Original cross section

This chapter covers the EWM calculations for the original cross section

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350\,mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 335MPa & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 9.169 \times 10^{-4}\,m^4 + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) :=$ 4 if $\psi = 1$; $\frac{8.2}{1.05 + \psi}$ if $1 > \psi > 0$; 7.81 if $\psi = 0$; $7.81 - 6.29\psi + 9.78\psi^2$ if $0 > \psi > -1$; 23.9 if $\psi = -1$; $5.98(1 - \psi)^2$ if $-1 > \psi > -3$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$k_{\sigma.4.2a}(\psi) :=$ 0.43 if $\psi = 1$; 0.57 if $\psi = 0$; 0.85 if $\psi = -1$; $0.57 - 0.21\psi + 0.07\psi^2$ if $-1 > \psi \geq -3$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2b}(\psi) :=$ 0.43 if $\psi = 1$; $\frac{0.578}{\psi + 0.34}$ if $1 > \psi > 0$; 1.7 if $\psi = 0$; $1.7 - 5\psi + 17.1\psi^2$ if $0 > \psi > -1$; 23.8 if $\psi = -1$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) :=$ 1.0 if $\lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi}$; $\min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right]$ otherwise

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) :=$ 1.0 if $\lambda_p \leq 0.748$; $\min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right)$ otherwise

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$Condition := 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s}$ if $\psi_s \leq -1$; $\frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s}$ otherwise

$Class4 :=$ "Class4" if $\frac{h_s}{t_s} > Condition$ = "Lower than class 4"; "Lower than class 4" otherwise

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1v} := t_d = 10 \cdot mm$

$z_{2v} := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$Class4_v :=$ "Class4" if $\frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})}$ = "Lower than class 4"; "Lower than class 4" otherwise

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.484$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m$ $\qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.31 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.053 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{5} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 639.127$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot (1 - \nu^2) \cdot b_d^{2}} = 990.256 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{p.} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 5.375 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{7} \cdot mm^4$

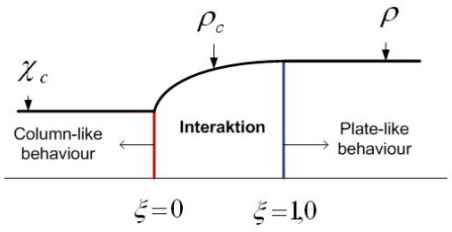
Buckling resistance stiffener

$A_{sl.} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.026 \times 10^{3} \cdot MPa$

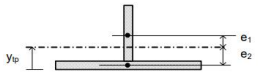
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is amost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -319.999 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 281.032 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.032 m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118 m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 371.724 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.788 \times 10^{-4} m^4$$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323 \times 10^{-3} m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015 m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{\delta f.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -5.041 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 432.173 \cdot cm^2$$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.094 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.916 \cdot mm$$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 390.532 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.778 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206\,mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.977 \times 10^{-3}\,m^4$$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 7.825 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.635 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.865 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 4.408 \times 10^{-4}\,m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.448 \times 10^{10}\,mm^4$$

Load

Load capacity:

$q_M := 92\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$$Q_M := q_M \cdot b_d = 322 \cdot \frac{kN}{m}$$

$$M_{Ed} := \frac{Q_M \cdot l_{bridge}^{2}}{8} = 1.304 \times 10^{4} \cdot kN \cdot m$$

$q_V := 134\,\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$$Q_V := q_V \cdot b_d = 469 \cdot \frac{kN}{m}$$

$$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.221 \times 10^{3} \cdot kN$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.037 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.316 \times 10^{4} \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.991$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15ε on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$$ Angle of web

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15ε each side of the stiffeners:

$$a_{stw.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15ε of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.455$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon\left(t_w\right)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot \text{kN}$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221 \times 10^3 \cdot \text{kN}$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.993$ (contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

S83 "bärande konstruktioner"

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 385.532 \cdot \text{mm}$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 491.468 \cdot \text{mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left|a_{öf.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} \ldots + \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y\left(t_b\right)}{\gamma_{M0}} = 7.191 \times 10^3 \cdot \text{kN} \cdot \text{m}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.623 \text{ m}$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59 \text{ m}$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 294.676 \cdot \text{kN}$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{kN}$$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot \text{kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.993$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.643 \times 10^{-3} \cdot \text{m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 432.579 \cdot \text{cm}^2$

$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \ldots + \left[n_{so} \cdot t_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 328.801 \cdot \text{mm}$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.026 \cdot \text{m}^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 9.332 \times 10^3 \cdot kN \cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 1.316 \times 10^4 \cdot kN \cdot m$$

Moment capacity check when $q_M = 92 \cdot \frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.991$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 134 \cdot \frac{kN}{m^2}$

$$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$$

The Reduced stress method

- Original cross section

Material

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ (335MPa) & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} := 1$

$E := 210GPa$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot GPa$

Geometry

Bottom plate

$t_b := 16mm$ $b_b := 1000mm$ $A_b := b_b\cdot t_b = 0.016\,m^2$

Top plate

$t_d := 10mm$ $b_d := 3500mm$ $A_d := b_d\cdot t_d = 0.035\,m^2$

Side plates

$t_s := 12mm$ $h_s := 389.2mm$ $A_s := h_s\cdot t_s = 4.67\times 10^{-3}\,m^2$

Webs

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w\cdot t_w = 0.016\,m^2$

$h_{w.x} := 1250mm$ $h_{w.y} := 482.8mm$

Top stiffeners

$t_{so} := 16mm$ $h_{so} := 140mm$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times 10^{-3}\,m^2$

$s_{so} := 350mm$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12mm$ $h_{su} := 120mm$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times 10^{-3}\,m^2$

$s_{su} := 440mm$ $n_{su} := 3$

$I_{tot} := 0.0148m^4$ From grasshopper

$Y_{tp} := 0.526956m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 78\frac{kN}{m^2}$

$Q := q\cdot b_d = 273\cdot\frac{kN}{m}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.106\times 10^4\cdot kN\cdot m$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.457\times 10^3\cdot kN$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 267.481\cdot MPa$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times 10^{-3}\cdot m^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 15.761\cdot MPa$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts} - t_d - Y_{tp}) = 263.745\cdot MPa$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 11.375\cdot MPa$

$S_{sp} := (h_{ts} - t_d - Y_{tp})\cdot t_s\cdot\frac{(h_{ts} - t_d - Y_{tp})}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) + \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right) = 9.867\times 10^{-3}\cdot m^3$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\,t_s} = 68.25\cdot MPa$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts} - t_d - Y_{tp}) = 263.745\cdot MPa$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) + \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.479\times 10^{-3}\cdot m^3$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 15.348\cdot MPa$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.35\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT}\cdot q = 339.3\cdot MPa$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.046$

$\alpha_{cr.UNIT} := 228.417\cdot\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.928$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.598$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\cdot MPa$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times 10^{-7}\,m^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}A_{so}} = 32.411\cdot mm$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right] = 1.052\times 10^8\cdot mm^4$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 639.127$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp} \cdot \sigma_{E.tp} = 990.256 \cdot MPa$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \frac{3 - \psi_{tp}}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$

$b_{2.sup.tp} := \frac{2}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$

$A_{sl.1.tp} := t_d \cdot b_{sl.1.tp} + A_{so} = 5.74 \times 10^{-3}\,m^2$

$y_{tp.sl.1.tp} := \frac{t_d \cdot b_{sl.1.tp} \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right)}{t_d \cdot b_{sl.1.tp} + A_{so}} = 0.034\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp} \cdot t_d^3}{12} + b_{sl.1.tp} \cdot t_d \cdot \left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{so} \cdot h_{so}^3}{12} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right)^2$ Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2 \cdot E \cdot I_{sl.1.tp}}{A_{sl.1.tp} \cdot s_{ts}^2} = 1.026 \times 10^3 \cdot MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,m$

$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d, t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.046\,m$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5 \cdot \left[1 + \alpha_{e.tp} \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.794$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1\right) = 0.759$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \quad = 0.759 \\ 1 & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \quad = 1.2 \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \leq \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \geq 1.08 \end{cases}$

$\frac{\rho_{c.tp} \cdot \alpha_{ult}}{\gamma_{M1}} = 0.794$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})} \cdot z + \sigma_{max}\right] \cdot -1$

$\psi_{sp.x} := \frac{\sigma(h_{ts} - t_d - h_s)}{\sigma(h_{ts} - t_d)} = -0.102$

$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - Y_{tp}) = 263.745 \cdot MPa$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - h_s - Y_{tp}) = -27.011 \cdot MPa$

$\psi_{sp3.x} := \max(3 + \psi_{sp.x}, 0) = 2.898$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \quad = 1 \\ \frac{\lambda_p - 0.055 \cdot (\psi_{sp3.x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases}$

Plate-like behaviour

$\sigma_{E.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 180.432 \cdot MPa$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \quad = 8.557 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp.x} + 9.78 \cdot \psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98 \cdot (1 - \psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases}$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 1.544 \times 10^3 \cdot MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 6.833 \cdot MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5 \cdot \left[1 + \alpha_{sp} \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.72$

$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1\right) = 0.891$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) \cdots + \chi_{sp}\right] & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) \cdots + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \quad = 1 \\ 1 & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34$ $\lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5 \cdot \left[1 + \alpha_{p.sp} \cdot (\lambda_p - \lambda_{p0.sp}) + \lambda_p\right] = 0.781$

$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \quad = 1 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases}$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{\,2}} = 6.833 \cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 27.331 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_s^{\,2}} = 180.432 \cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) \cdots + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) \cdots + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \quad = 0.8 \\ 1 \quad \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 \quad \text{if } \lambda_p \leq 0.748 \quad = 1 \\ \frac{\lambda_p - 0.188}{\lambda_p^{\,2}} \quad \text{if } \lambda_p > 0.748 \end{cases}$$

$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$

$\sigma_{E.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_{so}^{\,2}} = 2.479 \times 10^3 \cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 \quad \text{if } \psi_{tps.x} = 1 \quad = 4.117 \\ \frac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \quad \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 \quad \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{tps.x} + 9.78 \cdot \psi_{sp.x}^{\,2} \quad \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 \quad \text{if } \psi_{tps.x} = -1 \\ 5.98 \cdot \left(1 - \psi_{tps.x}\right)^2 \quad \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 \quad \text{if } \psi_{tps.x} < -3 \end{cases}$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x} \cdot \sigma_{E.tps.x} = 1.021 \times 10^4 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{\,2}} = 12.147 \cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) \cdots + \chi_{tps}\right] & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) \cdots + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 \quad \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp} \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp}}{\frac{\chi_w \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.99$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.604$$
$$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tps}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 = 0.556$$

C.2 Changed top plate thickness

CXIII 4 mm top plate EWM calculation.
CXX 4 mm top plate RSM calculation.
CXXIII 6 mm top plate EWM calculation.
CXXX 6 mm top plate RSM calculation.
CXXXIII 8 mm top plate EWM calculation.
CXL 8 mm top plate RSM calculation.
CXLIII 12 mm top plate EWM calculation.
CL 12 mm top plate RSM calculation.
CLIII 14 mm top plate EWM calculation.
CLX 14 mm top plate RSM calculation.
CLXIII 16 mm top plate EWM calculation.
CLXX 16 mm top plate RSM calculation.
CLXXIII 18 mm top plate EWM calculation.
CLXXX 18 mm top plate RSM calculation.
CLXXXIII 20 mm top plate EWM calculation.
CXC 20 mm top plate RSM calculation.

EWM - 4 mm top plate thickness

This chapter covers the EWM when changing the top plate thickness to 4 mm.

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 4mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 884 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.1 \cdot m^2$

$A_c := A_{so} + A_d = 0.034 m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 432.49 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 2.624 \times 10^{-3} m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 5.789 \times 10^{-4} m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp}\right]^2 = 1.938 \times 10^{-3} m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 2.595 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 3.148 \times 10^{-3} m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 6.289 \times 10^{-4} m^4$

Total:

$I_{v.tot} := I_{v.st.so} + I_{v.st.su} + I_{v.w} + I_{v.d} + I_{v.b} + I_{v.s} = 0.012 m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 4 \cdot mm$

$z_2 := t_d + h_s = 393.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.092$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 7.182$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.524$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = -0.039\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 428.49 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 428.49 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 171.396 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 257.094 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 1.028 \times 10^4 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.296 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1v} := t_d = 4 \cdot mm$

$z_{2v} := t_d + h_{so} = 144 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.673$

$k_{\sigma.4.2b}(\psi_{so}) = 0.57$ Higher stress close to support

$$Class4_v := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.501$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^4 \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 87.5 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -2 \times 10^{-3} m$ $\qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 1.807$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.486$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 162.346 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 3.334 \times 10^4 \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 2.658 \times 10^4 \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 2.051 \times 10^4 \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 44.492 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 7.578 \times 10^7 \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 3.694 \times 1 \qquad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.014 m^2$

$\delta := \frac{A_{sl}}{A_p} = 1.44$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 4.638 \times 10^3$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^2}{12 \cdot (1 - \nu^2) \cdot b_d^2} = 1.15 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.797$

$\lambda_p := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.496$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 2.723 \times 10^4 \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 9.667 \times 10^3 \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 46.308 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^3}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 8.127 \times 10^6 \cdot mm^4$

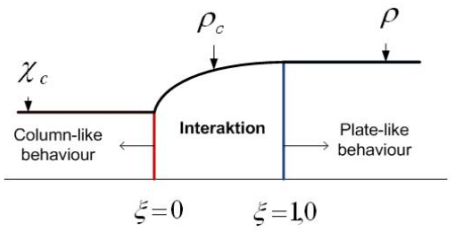
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 3.64 \times 10^3 \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 2.889 \times 10^3 \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.794$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.157 \times 10^3 \cdot MPa$

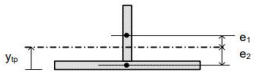
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.494$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_v := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 144.285 \cdot mm$

$e_c := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 42.492 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.517$

$\phi := 0.5\left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.698$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.84$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.84$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -7.997 \times 10^3 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} s_{so.eff} \cdot t_d + A_{so}} = -307.541 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 298.165 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} t_d \cdot s_{so.eff} = 6.494 \times 10^{-3} m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.093 m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 467.312 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 7.924 \times 10^{-4} m^4$$

$I_{y.d.eff} := \frac{n_{s.so} s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 1.406 \times 10^{-3} m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.011 m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 2.658 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -3.529 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was

$A_{c.eff.neg} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 229.743 \cdot cm^2$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 891.151 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 3.36 \cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 473.498 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 1.444 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206\,mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 3.25 \times 10^{-3}\,m^4$$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 4.362 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 1.459 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 2.592 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2\left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 8.238 \times 10^{-4}\,m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.001 \times 10^{10}\,m$

Load

Load capacity:

$q_M := 52\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$$Q_M := q_M \cdot b_d = 182 \cdot \frac{kN}{m}$$

$$M_{Ed} := \frac{Q_M \cdot l_{bridge}^{2}}{8} = 7.371 \times 10^{3} \cdot kN \cdot m$$

$q_V := 134\,\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$$Q_V := q_V \cdot b_d = 469 \cdot \frac{kN}{m}$$

$$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.221 \times 10^{3} \cdot kN$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.021 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 7.501 \times 10^{3} \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.983$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15ε on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$$ Angle of web

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15ε each side of the stiffeners:

$$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15ε of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.455$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 1.493$

SS-EN 1993-1-5 (A.

$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta := 1.2$ EC1993-1-3-5 ANM.2 5.1

$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon\left(t_w\right)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$

SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot \text{kN}$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221 \times 10^3 \cdot \text{kN}$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.993$ (contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated. S83 "bärande k

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 471.498 \cdot \text{mm}$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 402.502 \cdot \text{mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left|a_{öf.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y\left(t_b\right)}{\gamma_{M0}} = 3.373 \times 10^3 \cdot \text{kN} \cdot \text{m}$

The contribution from the flanges should be taken into account when there are still capacity left in the flan

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.52 \text{ m}$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59 \text{ m}$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 160.291 \cdot \text{kN}$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 \cdot \text{kN} \end{cases}$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot \text{kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.993$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 4.036 \times 10^{-3} \cdot \text{m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 220.419 \cdot \text{cm}^2$

$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + \left[n_{so} \cdot t_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 636.349 \cdot \text{mm}$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 6.343 \times 10^{-3} \cdot \text{m}^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 2.252 \times 10^3 \cdot kN \cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 7.501 \times 10^3 \cdot kN \cdot m$$

Moment capacity check when $q_M = 52 \cdot \frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.983$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 134 \cdot \frac{kN}{m^2}$

$$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$$

The Reduced stress method

- 4 mm cross section

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 4\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.014\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.884\,\text{m}$ $s_{ts} := 2\text{m}$

$I_{tot} := 0.011591\text{m}^4$ From grasshopper

$Y_{tp} := 0.452738\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 26\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 91.\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 3.686\times10^3\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 819\cdot\text{kN}$

$M_{Ed.end} := V_{Ed}\cdot s_{ts} - \frac{Q\cdot(s_{ts})^2}{2} = 1.456\times10^3\cdot\text{kN}\cdot\text{m}$

$V_{Ed.mid} := V_{Ed} - Q\cdot 8\text{m} = 91\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 136.489\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 5.463\times10^{-4}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 3.217\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts} - t_d - Y_{tp}) = 135.853\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 3.792\cdot\text{MPa}$

$S_{sp} := (h_{ts} - t_d - Y_{tp})\cdot t_s\cdot\frac{(h_{ts} - t_d - Y_{tp})}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 7.701\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\,t_s} = 22.674\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts} - t_d - Y_{tp}) = 135.853\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 9.657\times10^{-4}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 4.265\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 8.02\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

7.3792

$\sigma_V := \sigma_{V.UNIT}\cdot q = 208.52\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.702$

$\alpha_{cr.UNIT} := 28.110\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 1.081$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 1.255$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases}$ $= 0.657$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 0.248\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 2.051\times10^{-8}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\,A_{so}} = 0.044\,\text{m}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 3.694\times10^3$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 1.44$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha_{tp}^{\,2}\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}^{\,2}\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 4.638\times10^{3}$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 1.15\times10^{3}\cdot\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,\text{m}$

$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,\text{m}$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,\text{m}$

$A_{sl.1.tp} := t_d\cdot b_{sl.1.tp} + A_{so} = 3.64\times10^{-3}\,\text{m}^2$

$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\cdot b_{sl.1.tp}+A_{so}} = 0.046\,\text{m}$ combined center of gravity fo stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^{\,3}}{12} + b_{sl.1.tp}\cdot t_d\cdot\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \frac{t_{so}\cdot h_{so}^{\,3}}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$

Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}^{\,2}} = 1.157\times10^{3}\cdot\text{MPa}$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.047\,\text{m}$

$e_{tp} := \max\left(y_{tp.sl.1.tp}-t_d, t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right) = 0.042\,\text{m}$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.571$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\cdot\left[1+\alpha_{e.tp}\cdot\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 1.588$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}^{\,2}-\lambda_p^{\,2}}}, 1\right) = 0.39$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}}-1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases}$ = 0.3

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 0.701$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})}\cdot z + \sigma_{max}\right]\cdot -1$

$\psi_{sp.x} := \frac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = 0.089$

$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-Y_{tp}\right) = 135.853\cdot\text{MPa}$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-h_s-Y_{tp}\right) = 12.102\cdot\text{MPa}$

$\psi_{sp3.x} := \max\left(3+\psi_{sp.x}, 0\right) = 3.089$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = 0.089$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p-0.055\cdot\left(\psi_{sp3.x}\right)}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases}$ = 0.689

Plate-like behaviour

$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^2\right)\cdot h_s^{\,2}} = 180.432\cdot\text{MPa}$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \frac{8.2}{\left(1.05+\psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81-6.29\cdot\psi_{sp.x}+9.78\cdot\psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases}$ = 7.199

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.299\times10^{3}\cdot\text{MPa}$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^{\,2}} = 6.833\cdot\text{MPa}$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\cdot\left[1+\alpha_{sp}\cdot\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 1.398$

$\chi_{sp} := \min\left(\frac{1}{\phi_{sp}+\sqrt{\phi_{sp}^{\,2}-\lambda_p^{\,2}}}, 1\right) = 0.496$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}}-1, 0\right), 1\right) = 1$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) \cdots\right] + \chi_{sp} & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) \cdots\right] + \chi_{sp}}{\gamma_{M1}}\cdot\alpha_{ult} < 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} =$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\cdot\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 1.222$

$$\rho_{pz.sp} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} \text{ if } \lambda_p > 0.673 \end{cases} = 0.585$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1 - \nu^2\right)\cdot s_{ts}^2} = 6.833\cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 27.331\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1 - \nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] + \chi_{sp} & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] + \chi_{sp}}{\gamma_{M1}}\cdot\alpha_{ult} < 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} =$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} \text{ if } \lambda_p > 0.748 \end{cases} = 0.678$$

$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.672$

$\sigma_{E.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1 - \nu^2\right)\cdot h_{so}^2} = 2.479 \times 10^3\cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 \\ \frac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 \text{ if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 \text{ if } \psi_{tps.x} < -3 \end{cases} = 4.761$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.18 \times 10^4\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1 - \nu^2\right)\cdot s_{ts}^2} = 12.147\cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] + \chi_{tps} & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] + \chi_{tps}}{\gamma_{M1}}\cdot\alpha_{ult} \\ 1 \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.971$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.164$$
$$+ 3\cdot\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 = 0.147$$

EWM - 6 mm top plate thickness

This chapter covers the EWM when changing the top plate thickness to 6 mm.

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 6mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 886 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.107 \cdot m^2$

$A_c := A_{so} + A_d = 0.041\,m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 406.21 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 2.231 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 6.942 \times 10^{-4}\,m^4 + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp}\right]^2 = 2.331 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 3.414 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 3.562 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 5.128 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.013\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 6 \cdot mm$

$z_2 := t_d + h_s = 395.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.028$

$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$

$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$k_{\sigma.4.1}(\psi_s) = 7.61$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.509$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = -0.011\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 400.21 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 400.21 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 160.084 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 240.126 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 9.605 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.373 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1w} := t_d = 6 \cdot mm$

$z_{2w} := t_d + h_{so} = 146 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.65$

$k_{\sigma.4.2b}(\psi_{so}) = 0.584$ Higher stress close to support

$Class4_w := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.496$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 58.333 \quad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -3 \times 10^{-3} m$

$\psi_d := \psi_{neg}(z_d, z_d) = 1$

$\delta_{sso} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 1.205$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.679$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 226.638 \cdot mm$

Areas:

$A_{sso} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 3.992 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 3.326 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot (1 - \nu^2)} = 6.923 \times 10^{4} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 38.755 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 8.78 \times 10^{7} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 1.268 \times 1 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.021 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.96$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[4]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 1.983 \times 10^{3}$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot (1 - \nu^2) \cdot b_d^{2}} = 1.106 \times 10^{3} \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.833$

$\lambda_{p.} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.517$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 3.462 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.229 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 40.677 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 9.441 \times 10^{6} \cdot mm^4$

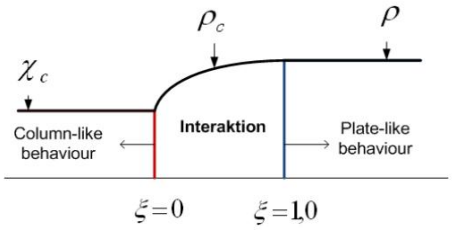
Buckling resistance stiffener

$A_{sl.} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 4.34 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 3.6 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.829$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.127 \times 10^{3} \cdot MPa$

Slenderness ratio

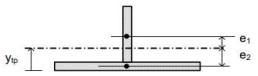
$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.511$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_{.} := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 142.237 \cdot mm$



$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 37.245 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.514$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.711$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.831$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is almost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.831$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.013 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -312.68 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 294.838 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.014\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.1\,m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 436.085 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 6.359 \times 10^{-4}\,m^4$$

$$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 2.551 \times 10^{-3}\,m^4$$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.013\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 3.326 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.092 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 289.853 \cdot cm^2$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 951.261 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 4.983 \cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 445.321 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 2.661 \times 10^{-3}\,m^4$$

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so}\cdot\frac{t_{so}\cdot h_{so.eff}^{3}}{12} + n_{so}\cdot t_{so}\cdot h_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.783\times 10^{-3}\,m^4$$

$I_{y.st.su.1.fin} := 6\cdot 1.506\cdot 10^{-6}\,m^4$ from FEM

$$\begin{aligned} I_{y.st.su.fin} := {} & I_{y.st.su.1.fin} \cdots \\ & + 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots \\ & + 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots \\ & + 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \end{aligned} = 5.384\times 10^{-4}\,m^4$$

Web:

$I_{y.w.1.fin} := 2\cdot 0.000312\,m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w\cdot h_w\cdot\left[h_{ts} - \frac{t_b}{2} - \frac{1}{2}\cdot\left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 1.801\times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b\cdot t_b^{3}}{12} + t_b\cdot b_b\cdot\left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 2.996\times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2\left[\begin{aligned} & \frac{t_s\cdot h_{s.eff.c1}^{3}}{12} + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ & + \frac{t_s\cdot h_{s.eff.c2}^{3}}{12} + t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ & + \frac{t_s\cdot h_{s.t}^{3}}{12} + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 6.773\times 10^{-4}\,m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.146\times 10^{10}\,mm^4$

Load

Load capacity:

$q_M := 64\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M\cdot b_d = 224\cdot\frac{kN}{m}$

$M_{Ed} := \frac{Q_M\cdot l_{bridge}^{2}}{8} = 9.072\times 10^{3}\cdot kN\cdot m$

$q_V := 134\,\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V\cdot b_d = 469\cdot\frac{kN}{m}$

$V_{Ed} := \frac{Q_V\cdot l_{bridge}}{2} = 4.221\times 10^{3}\cdot kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.026\cdot m^3$

$M_{Rd} := W_{y.eff}\cdot\frac{f_y(t_d)}{\gamma_{M1}} = 9.132\times 10^{3}\cdot kN\cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.993$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15ε on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2\cdot 15\cdot\varepsilon(t_w)\cdot t_w = 0.293\,m$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right)\cdot\frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{su0}\cdot\cos\left(\alpha_w\cdot\frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w\cdot\frac{\pi}{180}\right)\cdot h_{su}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{su}\cdot\cos\left(\alpha_w\cdot\frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{su}\cdot\cos\left(\alpha_w\cdot\frac{\pi}{180}\right) = 1.324\,m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2t_{su}\cdot h_{su}\cdot\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18\times 10^{-3}\,m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15ε each side of the stiffeners:

$a_{stw.z.1} := \frac{b_b}{2} + s_{su0}\cdot\cos\left(\alpha_w\cdot\frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$

$a_{stw.z.2} := a_{st.z.1} + s_{su}\cdot\cos\left(\alpha_w\cdot\frac{\pi}{180}\right) = 0.935\,m$

$a_{stw.z.3} := a_{st.z.2} + s_{su}\cdot\cos\left(\alpha_w\cdot\frac{\pi}{180}\right) = 1.346\,m$

Moment of inirtia of 15ε of the web z-direction

$I_{stw.z} := 2b_{web}\cdot t_w\cdot\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\cdot m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00\cdot\left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34\cdot\left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4\cdot t_s\cdot\varepsilon(t_s)\cdot\sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon\left(t_w\right)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ S83 "bärande konstruktioner"

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot \text{kN}$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221 \times 10^3 \cdot \text{kN}$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.993$ (Contribution from two web)

$$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

S83 "bärande konstruktioner"

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 442.321 \cdot \text{mm}$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 432.679 \cdot \text{mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left|a_{öf.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} \ldots + \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y\left(t_b\right)}{\gamma_{M0}} = 4.593 \times 10^3 \cdot \text{kN} \cdot \text{m}$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.544\,\text{m}$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59\,\text{m}$$

$$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 196.906 \cdot \text{kN}$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{kN}$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot \text{kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.993$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 5.656 \times 10^{-3} \cdot \text{m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 282.418 \cdot \text{cm}^2$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \ldots + \left[n_{so} \cdot t_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 498.523 \cdot \text{mm}$$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.011 \cdot \text{m}^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 4.028 \times 10^3 \cdot kN \cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 9.132 \times 10^3 \cdot kN \cdot m$$

Moment capacity check when $q_M = 64 \cdot \frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.993$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 134 \cdot \frac{kN}{m^2}$

$$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$$

The Reduced stress method

-6 mm top plate

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 6\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.021\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.886\,\text{m}$

$s_{vc} := 2\text{m}$

$I_{tot} := 0.012799\text{m}^4$ From grasshopper

$Y_{tp} := 0.480699\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 45\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 157.5\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 6.379\times10^3\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 1.417\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 200.498\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 7.68\times10^{-4}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 7.088\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts} - t_d - Y_{tp}) = 199.003\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 6.563\cdot\text{MPa}$

$S_{sp} := (h_{ts} - t_d - Y_{tp})\cdot t_s\cdot\frac{(h_{ts} - t_d - Y_{tp})}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 8.5\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\,t_s} = 39.225\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts} - t_d - Y_{tp}) = 199.003\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.158\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 8.012\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 5.75\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT}\cdot q = 258.75\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.372$

$\alpha_{cr.UNIT} := 73.451\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 1.632$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.917$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.829$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 0.558\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 6.923\times10^{-8}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\cdot A_{so}} = 0.039\,\text{m}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 1.268\times10^3$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.96$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 1.983\times10^3$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 1.106\times10^3\cdot\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \frac{3 - \psi_{tp}}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$

$b_{2.sup.tp} := \frac{2}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$

$y_{tp.sl.1.tp} := \frac{t_d \cdot b_{sl.1.tp} \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right)}{t_d \cdot b_{sl.1.tp} + A_{so}} = 0.041\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp} \cdot t_d^{3}}{12} + b_{sl.1.tp} \cdot t_d \cdot \left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{so} \cdot h_{so}^{3}}{12} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right)^2$ Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2 \cdot E \cdot I_{sl.1.tp}}{A_{sl.1.tp} \cdot s_{ts}^{2}} = 1.127 \times 10^{3} \cdot MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.047\,m$

$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d, t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.035\,m$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{to}}} = 0.558$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5 \cdot \left[1 + \alpha_{e.tp} \cdot (\lambda_p - 0.2) + \lambda_p^{2}\right] = 1.12$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^{2} - \lambda_p^{2}}}, 1\right) = 0.567$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \frac{\left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.567$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \leq \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \geq 1.08 \end{cases} = 0.905$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})} \cdot z + \sigma_{max}\right]^{-1}$

$\psi_{sp.x} := \frac{\sigma(h_{ts} - t_d - h_s)}{\sigma(h_{ts} - t_d)} = 0.025$

$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - Y_{tp}) = 199.003 \cdot MPa$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - h_s - Y_{tp}) = 5.034 \cdot MPa$

$\psi_{sp3.x} := \max(3 + \psi_{sp.x}, 0) = 3.025$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = 0.025$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{\lambda_p - 0.055 \cdot (\psi_{sp3.x})}{\lambda_p^{2}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.893$

Plate-like behaviour

$\sigma_{E.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{2}}{12 \cdot (1 - \nu^2) \cdot h_s^{2}} = 180.432 \cdot MPa$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp.x} + 9.78 \cdot \psi_{sp.x}^{2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98 \cdot (1 - \psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 7.626$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 1.376 \times 10^{3} \cdot MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{2}} = 6.833 \cdot MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5 \cdot \left[1 + \alpha_{sp} \cdot (\lambda_p - 0.2) + \lambda_p^{2}\right] = 0.996$

$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^{2} - \lambda_p^{2}}}, 1\right) = 0.723$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) \ldots + \chi_{sp}\right] & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) \ldots + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34$ $\lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5 \cdot \left[1 + \alpha_{p.sp} \cdot (\lambda_p - \lambda_{p0.sp}) + \lambda_p\right] = 0.995$

$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^{2} - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.789$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{\,2}} = 6.833 \cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 27.331 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_s^{\,2}} = 180.432 \cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{sp} & \\ 1 & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.7$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.748 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.748 \end{cases} = 0.867$$

$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.649$

$\sigma_{E.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_{so}^{\,2}} = 2.479 \times 10^3 \cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 & \text{if } \psi_{tps.x} = 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{tps.x} + 9.78 \cdot \psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{tps.x} = -1 \\ 5.98 \cdot \left(1 - \psi_{tps.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{tps.x} < -3 \end{cases} = 4.825$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x} \cdot \sigma_{E.tps.x} = 1.196 \times 10^4 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{\,2}} = 12.147 \cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{tps} & \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp} \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp}}{\frac{\chi_w \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.994$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.345$$

$$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

EWM - 8 mm top plate thickness

This chapter covers the EWM when changing the top plate thickness to 8 mm.

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 8mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 888 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.114 \cdot m^2$

$A_c := A_{so} + A_d = 0.048 m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 383.272 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.912 \times 10^{-3} m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 8.073 \times 10^{-4} m^4 + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp}\right]^2 = 2.721 \times 10^{-3} m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.028 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 3.948 \times 10^{-3} m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 4.228 \times 10^{-4} m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.014 m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 8 \cdot mm$

$z_2 := t_d + h_s = 397.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.037$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.057$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.495$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.014\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 375.272 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 375.272 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 150.109 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 225.163 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 9.007 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.454 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1w} := t_d = 8 \cdot mm$

$z_{2w} := t_d + h_{so} = 148 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.627$

$k_{\sigma.4.2b}(\psi_{so}) = 0.598$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.49$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^{2}$

Deck

$\frac{s_{so}}{t_d} = 43.75 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -4 \times 10^{-3} m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.903$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.837$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 279.676 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 4.651 \times 10^{4} \cdot mm^{2}$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 4.145 \times 10^{4} \cdot mm^{2}$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot (1 - \nu^{2})} = 1.641 \times 10^{5} \cdot mm^{4}$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 34.977 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^{2}\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^{2}\right] = 9.726 \times 10^{7} \cdot mm^{4}$

$\gamma := \frac{I_{sl}}{I_p} = 592.685 \qquad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^{2}$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.028 m^{2}$

$\delta := \frac{A_{sl}}{A_p} = 0.72$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^{2}\right)^{2} + \gamma - 1\right]}{\alpha^{2} \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 1.057 \times 10^{3}$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^{2} \cdot t_d^{2}}{12 \cdot (1 - \nu^{2}) \cdot b_d^{2}} = 1.048 \times 10^{3} \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.891$

$\lambda_p := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.549$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 4.369 \times 10^{4} \cdot mm^{2}$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.551 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 36.889 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^{2}\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^{2}\right] = 1.049 \times 10^{7} \cdot mm^{4}$

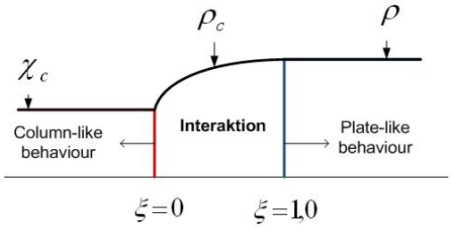
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.04 \times 10^{3} \cdot mm^{2}$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 4.477 \times 10^{3} \cdot mm^{2}$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.888$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^{2} \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^{2}} = 1.078 \times 10^{3} \cdot MPa$

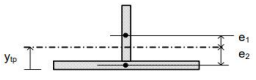
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.541$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 138.917 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 43.023 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.518$

$\phi := 0.5\left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.734$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.812$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is almost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.812$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.277 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -317.005 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_b)}{\gamma_M} = 288.272 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.022\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.109\,m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 402.906 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 4.927 \times 10^{-4}\,m^4$$

$$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 3.56 \times 10^{-3}\,m^4$$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.014\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 4.145 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.583 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 358.952 \cdot cm^2$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.02 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 6.496 \cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 416.802 \cdot mm$$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 3.813 \times 10^{-3}\,m^4$

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.347 \times 10^{-3}\,m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} + 2t_{su} \cdot h_{su} \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg} \right]^2 + 2t_{su} \cdot h_{su} \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg} \right]^2 + 2t_{su} \cdot h_{su} \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg} \right]^2 = 6.577 \times 10^{-4} m^4$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg} \right]^2 = 2.206 \times 10^{-3} m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2} \right)^2 = 3.433 \times 10^{-3} m^4$$

Side plate:

$$I_{y.s.fin} := 2 \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg} \right)^2 + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg} \right)^2 + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg} \right)^2 \right] = 5.465 \times 10^{-4} m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.3 \times 10^{10} \cdot mm^4$

Load

Load capacity:

$q_M := 78 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_d = 273 \cdot \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.106 \times 10^4 \cdot kN \cdot m$

$q_V := 134 \frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V \cdot b_d = 469 \cdot \frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.221 \times 10^3 \cdot kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.031 \cdot m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.108 \times 10^4 \cdot kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.998$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15ε on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293 m$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503 m$

$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913 m$

$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324 m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2 \right] = 8.18 \times 10^{-3} m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15ε each side of the stiffeners:

$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525 m$

$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935 m$

$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346 m$

Moment of inirtia of 15ε of the web z-direction

$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2 \right] = 0.021 \cdot m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029 m^4$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min(t_s, t_w)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min(t_s, t_w)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}} \right] = 4.782 \times 10^3$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18\cdot\frac{I_{st.z.tot}}{\min(t_s,t_w)^3\cdot h_{web}}}{\alpha^2} + 2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min(t_s,t_w)^3\cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635\times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s,t_w)} \leq 31\cdot\frac{\varepsilon(t_s)}{\eta}\cdot\sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w\cdot\varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7+\lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_{Mw} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ S83 "bärande konstruktioner"

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w\cdot f_y(t_s)\cdot h_{web}\cdot t_s}{\sqrt{3}\cdot\gamma_{M1}\cdot\gamma_n} = 2.125\times 10^3\cdot kN$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221\times 10^3\cdot kN$

$\frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$ (contribution from two web)

$$V_{b.Rd} \leq \frac{\eta\cdot f_y(t_s)\cdot h_{web}\cdot t_s}{\sqrt{3}\cdot\gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

S83 "bärande konstruktioner"

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 412.802\cdot mm$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 463.198\cdot mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg}\cdot|a_{öf.eff.neg}|\cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg}\cdot|a_{uf.eff.neg}|\cdot f_y(t_b)}{\gamma_{M0}} = 5.91\times 10^3\cdot kN\cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{öf} := a\cdot\left[0.25 + \frac{1.6\cdot b_d\cdot t_d^2\cdot f_y(t_d)}{t_s\cdot(h_s+h_{wy})^2\cdot f_y(t_s)}\right] = 0.579\,m$$

$$c_{uf} := a\cdot\left[0.25 + \frac{1.6\cdot b_b\cdot t_b^2\cdot f_y(t_b)}{t_s\cdot(h_s+h_{wy})^2\cdot f_y(t_s)}\right] = 0.59\,m$$

$$V_{bf.Rd.ored} := \frac{b_d\cdot t_d^2\cdot f_y(t_d)}{c_{öf}\cdot\gamma_{M1}\cdot\gamma_n} + \frac{b_b\cdot t_b^2\cdot f_y(t_b)}{c_{uf}\cdot\gamma_{M1}\cdot\gamma_n} = 242.948\cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored}\cdot\left[1-\left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta\cdot f_y(t_s)\cdot h_{web}\cdot t_s}{\sqrt{3}\cdot\gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125\times 10^3\cdot kN$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2V_{b.Rd}} = 0.993$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 7.246\times 10^{-3}\cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff}\cdot t_d + n_{so}\cdot s_{so.eff}\cdot t_{d.eff} + n_{so}\cdot t_{so}\cdot t_{d.eff} + A_b = 355.246\cdot cm^2$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) \ldots + \left[n_{so}\cdot t_{so}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) + A_b\cdot\left(h_{ts}-\frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 398.178\cdot mm$$

$$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.018\cdot m^3$$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 6.46\times 10^3\cdot kN\cdot m$$

Results

$M_{pl.Rd} := M_{Rd} = 1.108\times 10^4\cdot kN\cdot m$

Moment capacity check when $q_M = 78\cdot\frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.998$$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 134 \cdot \frac{kN}{m^2}$

$$\eta_{l3} := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$$

The Reduced stress method

-8 mm top plate thickness

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 8\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.028\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.888\,\text{m}$ $s_{ts} := 2\text{m}$

$I_{tot} := 0.013861\text{m}^4$ From grasshopper

$Y_{tp} := 0.505242\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 63\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 220.5\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 8.93\times10^3\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 1.984\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 244.023\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 9.641\times10^{-4}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 11.503\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts} - t_d - Y_{tp}) = 241.446\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 9.188\cdot\text{MPa}$

$S_{sp} := (h_{ts} - t_d - Y_{tp})\cdot t_s\cdot\frac{(h_{ts} - t_d - Y_{tp})}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.217\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\cdot t_s} = 54.985\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 241.446\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.327\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 11.877\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.93\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT}\cdot q = 310.59\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.143$

$\alpha_{cr.UNIT} := 141.726\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.25$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.713$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.97$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 0.992\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 1.641\times10^{-7}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}A_{so}} = 0.035\,\text{m}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 592.685$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.72$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 1.057\times10^3$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 1.048\times10^3\cdot\text{MPa}$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3 - \psi_{tp}}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$$

$$b_{2.sup.tp} := \frac{2}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$$

$$y_{tp.sl.1.tp} := \frac{t_d \cdot b_{sl.1.tp} \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right)}{t_d \cdot b_{sl.1.tp} + A_{so}} = 0.037\,m$$

combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp} \cdot t_d^{\,3}}{12} + b_{sl.1.tp} \cdot t_d \cdot \left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{so} \cdot h_{so}^{\,3}}{12} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2 \cdot E \cdot I_{sl.1.tp}}{A_{sl.1.tp} \cdot s_{ts}^{\,2}} = 1.078 \times 10^3 \cdot MPa$$

critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.046\,m$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d, t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.041\,m$$

$$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.571$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5 \cdot \left[1 + \alpha_{e.tp} \cdot (\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.9$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^{\,2} - \lambda_p^{\,2}}}, 1\right) = 0.689$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \frac{\left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp}) \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.689$$

Shear

$$\eta := 1.2$$

7.1 (2)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \leq \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \geq 1.08 \end{cases} = 1.164$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})} \cdot z + \sigma_{max}\right] \cdot -1$$

$$\psi_{sp.x} := \frac{\sigma(h_{ts} - t_d - h_s)}{\sigma(h_{ts} - t_d)} = -0.039$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - Y_{tp}) = 241.446 \cdot MPa$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - h_s - Y_{tp}) = -9.305 \cdot MPa$$

$$\psi_{sp3.x} := \max(3 + \psi_{sp.x}, 0) = 2.961 \qquad \psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.039$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{\lambda_p - 0.055 \cdot (\psi_{sp3.x})}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.082$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_s^{\,2}} = 180.432 \cdot MPa$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp.x} + 9.78 \cdot \psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98 \cdot (1 - \psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.067$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 1.456 \times 10^3 \cdot MPa$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{\,2}} = 6.833 \cdot MPa$$

$$\alpha_{sp} := 0.21$$

$$\phi_{sp} := 0.5 \cdot \left[1 + \alpha_{sp} \cdot (\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.808$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^{\,2} - \lambda_p^{\,2}}}, 1\right) = 0.842$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \frac{\left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) \cdots + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) \cdots + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp}) \xi_{sp.x} \cdot (2 - \xi_{sp.x}) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Z-direction

$$\psi_{sp.z} := 1$$

$$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$$

$$\phi_{p.sp} := 0.5 \cdot \left[1 + \alpha_{p.sp} \cdot (\lambda_p - \lambda_{p0.sp}) + \lambda_p\right] = 0.859$$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^{\,2} - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.986$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 6.833 \cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 27.331 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 180.432 \cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] + \chi_{sp} & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] + \chi_{sp}}{\gamma_{M1}} \cdot \alpha_{ult} < 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.8$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} \text{ if } \lambda_p > 0.748 \end{cases} = 1$$

$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.626$

$\sigma_{E.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^2}{12 \cdot (1 - \nu^2) \cdot h_{so}^2} = 2.479 \times 10^3 \cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 \\ \frac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{tps.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{tps.x} + 9.78 \cdot \psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{tps.x} = -1 \\ 5.98 \cdot \left(1 - \psi_{tps.x}\right)^2 \text{ if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 \text{ if } \psi_{tps.x} < -3 \end{cases} = 3.884$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x} \cdot \sigma_{E.tps.x} = 9.629 \times 10^3 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 12.147 \cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] + \chi_{tps} & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] + \chi_{tps}}{\gamma_{M1}} \cdot \alpha_{ult} < 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp} \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp}}{\frac{\chi_w \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.997$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.496$$
$$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tps}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 = 0.465$$

EWM - 12 mm top plate thickness

This chapter covers the EWM when changing the top plate thickness to 12 mm.

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 12mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 892 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.128 \cdot m^2$

$A_c := A_{so} + A_d = 0.062 m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 345.231 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.43 \times 10^{-3} m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 1.022 \times 10^{-3} m^4 + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp}\right]^2 = 3.468 \times 10^{-3} m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.834 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.645 \times 10^{-3} m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 2.974 \times 10^{-4} m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.016 m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 12 \cdot mm$

$z_2 := t_d + h_s = 401.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.168$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 9.142$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.464$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.056\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 333.231 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 333.231 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 133.293 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 199.939 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 7.998 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.65 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1w} := t_d = 12 \cdot mm$

$z_{2w} := t_d + h_{so} = 152 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.58$

$k_{\sigma.4.2b}(\psi_{so}) = 0.628$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.478$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 29.167 \quad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$z_d := \frac{-t_d}{2} = -6 \times 10^{-3} m$ $\psi_d := \psi_{neg}(z_d, z_d) = 1$

$\delta_{sso} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.602$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 1$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 334 \cdot mm$

Areas:

$A_{sso} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.969 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.796 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot (1 - \nu^2)} = 5.538 \times 10^{5} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 30.649 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \ldots + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.121 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 202.422 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.042 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.48$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 420.434$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot (1 - \nu^2) \cdot b_d^{2}} = 938.039 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.971$

$\lambda_{p.} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.606$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 6.197 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 2.2 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 32.435 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \ldots + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.215 \times 10^{7} \cdot mm^4$

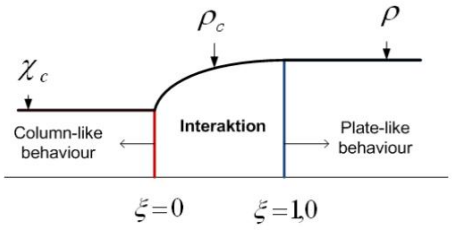
Buckling resistance stiffener

$A_{sl.} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 6.44 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 6.248 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.97$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 977.336 \cdot MPa$

Slenderness ratio

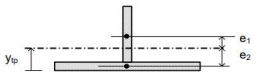
$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.594$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_{} := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 131.941 \cdot mm$



$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 51.351 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.525$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.78$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.778$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is alost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.778$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.804 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -320.812 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 276.311 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.04\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.126\,m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 350.385 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.11 \times 10^{-4}\,m^4$$

$$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.754 \times 10^{-3}\,m^4$$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.016\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.796 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -5.547 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 491.207 \cdot cm^2$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.153 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 9.34 \cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 372.22 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.376 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206\,mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.731 \times 10^{-3}\,m^4$$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 8.808 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.975 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 4.191 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 3.741 \times 10^{-4}\,m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.553 \times 10^{10}\,mm^4$

Load

Load capacity:

$q_M := 104\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$$Q_M := q_M \cdot b_d = 364 \cdot \frac{kN}{m}$$

$$M_{Ed} := \frac{Q_M \cdot l_{bridge}^{2}}{8} = 1.474 \times 10^{4} \cdot kN \cdot m$$

$q_V := 134\,\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$$Q_V := q_V \cdot b_d = 469 \cdot \frac{kN}{m}$$

$$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.221 \times 10^{3} \cdot kN$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.042 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.481 \times 10^{4} \cdot kN \cdot m$$

$\frac{M_{Ed}}{M_{Rd}} = 0.995$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon\left(t_w\right) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$$ Angle of web

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$$a_{stw.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15st of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon\left(t_s\right) \cdot \sqrt{k_{\tau 1}}} = 0.455$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon\left(t_w\right)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$

SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot \text{kN}$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221 \times 10^3 \cdot \text{kN}$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.993$ (contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

S83 "bärande konstruktioner"

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 366.22 \cdot \text{mm}$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 511.78 \cdot \text{mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left|a_{öf.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y\left(t_b\right)}{\gamma_{M0}} = 8.118 \times 10^3 \cdot \text{kN} \cdot \text{m}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.677 \text{ m}$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59 \text{ m}$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 348.726 \cdot \text{kN}$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{kN}$$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot \text{kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.993$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 9.567 \times 10^{-3} \cdot \text{m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 494.293 \cdot \text{cm}^2$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) \dots + \left[n_{so}\cdot t_{so}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) + A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 289.412\cdot mm$$

$$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.033\cdot m^3$$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 1.174\times 10^4\cdot kN\cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 1.481\times 10^4\cdot kN\cdot m$$

Moment capacity check when $q_M = 104\cdot\frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.995$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 134\cdot\frac{kN}{m^2}$

$$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$$

The Reduced stress method

-12 mm top plate thickness

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 12\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.042\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.892\,\text{m}$ $s_{ts} := 2\text{m}$

$I_{tot} := 0.015637\text{m}^4$ From grasshopper

$Y_{tp} := 0.546305\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 92\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 322\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.304\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.898\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 283.3\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.297\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 20.031\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 278.296\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 13.417\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 0.01\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\,t_s} = 80.771\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 278.296\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.616\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 18.722\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 3.98\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

3.3799

$\sigma_V := \sigma_{V.UNIT}\cdot q = 366.16\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 0.97$

$\alpha_{cr.UNIT} := 329.844\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 3.585$ =

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.52$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases}$ = 1

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 2.231\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 5.538\times10^{-7}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\,A_{so}} = 0.031\,\text{m}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 202.422$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.48$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases}$ = 420.434

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 938.039\cdot\text{MPa}$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3 - \psi_{tp}}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$$

$$b_{2.sup.tp} := \frac{2}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$$

$$y_{tp.sl.1.tp} := \frac{t_d \cdot b_{sl.1.tp} \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right)}{t_d \cdot b_{sl.1.tp} + A_{so}} = 0.032\,m$$

combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp} \cdot t_d^{\,3}}{12} + b_{sl.1.tp} \cdot t_d \cdot \left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{so} \cdot h_{so}^{\,3}}{12} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2 \cdot E \cdot I_{sl.1.tp}}{A_{sl.1.tp} \cdot s_{ts}^{\,2}} = 977.336 \cdot MPa$$

critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.043\,m$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d, t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.05\,m$$

$$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{to}}} = 0.593$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5 \cdot \left[1 + \alpha_{e.tp} \cdot (\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.73$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^{\,2} - \lambda_p^{\,2}}}, 1\right) = 0.805$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp} \cdot \left(2 - \xi_{tp}\right) + \chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp} \cdot \left(2 - \xi_{tp}\right) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp} \cdot \left(2 - \xi_{tp}\right) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.805$$

Shear

$$\eta := 1.2 \qquad 7.1\ (2)$$

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \leq \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \geq 1.08 \end{cases} = 1.2$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})} \cdot z + \sigma_{max}\right] \cdot -1$$

$$\psi_{sp.x} := \frac{\sigma\left(h_{ts} - t_d - h_s\right)}{\sigma\left(h_{ts} - t_d\right)} = -0.166$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}} \cdot \left(h_{ts} - t_d - Y_{tp}\right) = 278.296 \cdot MPa$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}} \cdot \left(h_{ts} - t_d - h_s - Y_{tp}\right) = -46.29 \cdot MPa$$

$$\psi_{sp3.x} := \max\left(3 + \psi_{sp.x}, 0\right) = 2.834 \qquad \psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.166$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{\lambda_p - 0.055 \cdot \left(\psi_{sp3.x}\right)}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot \left(1 - \nu^2\right) \cdot h_s^{\,2}} = 180.432 \cdot MPa$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp.x} + 9.78 \cdot \psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98 \cdot \left(1 - \psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 9.127$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 1.647 \times 10^3 \cdot MPa$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot \left(1 - \nu^2\right) \cdot s_{ts}^{\,2}} = 6.833 \cdot MPa$$

$$\alpha_{sp} := 0.21$$

$$\phi_{sp} := 0.5 \cdot \left[1 + \alpha_{sp} \cdot (\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.669$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^{\,2} - \lambda_p^{\,2}}}, 1\right) = 0.918$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) \cdots \right. \\ \left. + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) \cdots \right. \\ \left. + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Z-direction

$$\psi_{sp.z} := 1$$

$$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$$

$$\phi_{p.sp} := 0.5 \cdot \left[1 + \alpha_{p.sp} \cdot \left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.729$$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^{\,2} - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{\,2}} = 6.833 \cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 27.331 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_s^{\,2}} = 180.432 \cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) \cdots + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) \cdots + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \quad = 0.9 \\ 1 \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.748 & = 1 \\ \frac{\lambda_p - 0.188}{\lambda_p^{\,2}} \text{ if } \lambda_p > 0.748 \end{cases}$$

$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.58$

$\sigma_{E.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_{so}^{\,2}} = 2.479 \times 10^3 \cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 & = 4.43 \\ \frac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{tps.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{tps.x} + 9.78 \cdot \psi_{sp.x}^{\,2} \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{tps.x} = -1 \\ 5.98 \cdot \left(1 - \psi_{tps.x}\right)^2 \text{ if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 \text{ if } \psi_{tps.x} < -3 \end{cases}$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x} \cdot \sigma_{E.tps.x} = 1.098 \times 10^4 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{\,2}} = 12.147 \cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tns.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) \cdots + \chi_{tps}\right] & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) \cdots + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp} \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp}}{\frac{\chi_w \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.99$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.692$$

$$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tps}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 = 0.62$$

EWM - 14mm top plate thickness

This chapter covers the EWM when changing the top plate thickness to 14mm.

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 14mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 894 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.135 \cdot m^2$

$A_c := A_{so} + A_d = 0.069 m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 329.319 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.246 \times 10^{-3} m^4$$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 1.123 \times 10^{-3} m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 3.821 \times 10^{-3} m^4$$

Deck:

$$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 5.091 \times 10^{-3} m^4$$

Bottom flange:

$$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.959 \times 10^{-3} m^4$$

Side plate:

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 14 \cdot mm$

$z_2 := t_d + h_s = 403.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.234$

$\text{Condition} := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$

$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$k_{\sigma.4.1}(\psi_s) = 9.821$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.448$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.074\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 315.319 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 315.319 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 126.127 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 189.191 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 7.568 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.772 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1w} := t_d = 14 \cdot mm$

$z_{2w} := t_d + h_{so} = 154 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.556$

$k_{\sigma.4.2b}(\psi_{so}) = 0.645$ Higher stress close to support

$\text{Class4}_w := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.471$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 25 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$z_d := \frac{-t_d}{2} = -7 \times 10^{-3} m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$\delta_{so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.516$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 1$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 334 \cdot mm$

Areas:

$A_{ex} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 6.628 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 6.426 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot (1 - \nu^2)} = 8.795 \times 10^{5} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 29.445 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.184 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 134.641 \qquad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.049 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.411$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 293.79$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot (1 - \nu^2) \cdot b_d^{2}} = 892.181 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.97$

$\lambda_{p.} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.621$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 6.894 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 2.447 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 31.157 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.285 \times 10^{7} \cdot mm^4$

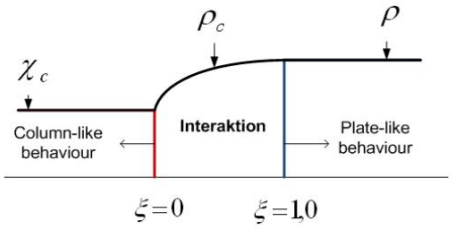
Buckling resistance stiffener

$A_{sl.} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 7.14 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 6.916 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.969$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 932.756 \cdot MPa$

Slenderness ratio

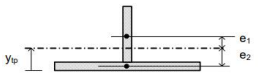
$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.607$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_{.} := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 128.782 \cdot mm$



$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 54.555 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.528$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.792$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.769$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is almost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.769$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.995 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -320.57 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 273.048 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.047 \, m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.133 \, m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 334.745 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 2.665 \times 10^{-4} \, m^4$$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.024 \times 10^{-3} \, m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.017 \, m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 6.426 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -6.082 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 541.015 \cdot cm^2$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.202 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 10.768 \cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 358.511 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.778 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206\,mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.552 \times 10^{-3}\,m^4$$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 9.617 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 3.256 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 4.452 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 3.278 \times 10^{-4}\,m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.633 \times 10^{10}\,mm^4$

Load

Load capacity:

$q_M := 114\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$$Q_M := q_M \cdot b_d = 399 \cdot \frac{kN}{m}$$

$$M_{Ed} := \frac{Q_M \cdot l_{bridge}^{2}}{8} = 1.616 \times 10^{4} \cdot kN \cdot m$$

$q_V := 134\,\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$$Q_V := q_V \cdot b_d = 469 \cdot \frac{kN}{m}$$

$$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.221 \times 10^{3} \cdot kN$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.046 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.617 \times 10^{4} \cdot kN \cdot m$$

$\frac{M_{Ed}}{M_{Rd}} = 0.999$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$$ Angle of web

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$$

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$$a_{stw.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15st of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.455$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon\left(t_w\right)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$

SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot \text{kN}$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221 \times 10^3 \cdot \text{kN}$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.993$ (contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

S83 "bärande konstruktioner"

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 351.511 \cdot \text{mm}$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 527.489 \cdot \text{mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left|a_{öf.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y\left(t_b\right)}{\gamma_{M0}} = 8.831 \times 10^3 \cdot \text{kN} \cdot \text{m}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.741 \text{ m}$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59 \text{ m}$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 402.44 \cdot \text{kN}$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{kN}$$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot \text{kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.993$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.01 \cdot \text{m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 545.954 \cdot \text{cm}^2$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) \dots + \left[n_{so}\cdot t_{so}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) + A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 263.6\cdot mm$$

$$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.039\cdot m^3$$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 1.378\times 10^4\cdot kN\cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 1.617\times 10^4\cdot kN\cdot m$$

Moment capacity check when $q_M = 114\cdot\frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.999$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 134\cdot\frac{kN}{m^2}$

$$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$$

The Reduced stress method

-14mm top plate thickness t_d

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 14\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.049\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.894\,\text{m}$ $s_{ts} := 2\text{m}$

$I_{tot} := 0.016389\,\text{m}^4$ From grasshopper

$Y_{tp} := 0.563655\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 99\,\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 346.5\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.403\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 3.119\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 276.867\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.44\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 22.839\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 270.874\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 14.438\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 0.011\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\,t_s} = 87.256\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 270.874\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.741\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 20.706\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 3.6\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT}\cdot q = 356.4\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 0.996$

$\alpha_{cr.UNIT} := 334.902\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 3.383$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.543$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 3.037\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 8.795\times10^{-7}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\,A_{so}} = 0.029\,\text{m}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 134.641$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.411$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 293.79$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\,\sigma_{E.tp} = 892.181\cdot\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \frac{3 - \psi_{tp}}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$

$b_{2.sup.tp} := \frac{2}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$

$y_{tp.sl.1.tp} := \frac{t_d \cdot b_{sl.1.tp} \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right)}{t_d \cdot b_{sl.1.tp} + A_{so}} = 0.031\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp} \cdot t_d^{\,3}}{12} + b_{sl.1.tp} \cdot t_d \cdot \left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{so} \cdot h_{so}^{\,3}}{12} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right)^2$ Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2 \cdot E \cdot I_{sl.1.tp}}{A_{sl.1.tp} \cdot s_{ts}^{\,2}} = 932.756 \cdot MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.042\,m$

$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d, t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.053\,m$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{to}}} = 0.602$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5 \cdot \left[1 + \alpha_{e.tp} \cdot (\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.75$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^{\,2} - \lambda_p^{\,2}}}, 1\right) = 0.788$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \frac{\left[(\rho_{px.tp} - \chi_{tp})\xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp})\xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp})\xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.788$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \leq \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \geq 1.08 \end{cases} = 1.2$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})} \cdot z + \sigma_{max}\right] \cdot -1$

$\psi_{sp.x} := \frac{\sigma(h_{ts} - t_d - h_s)}{\sigma(h_{ts} - t_d)} = -0.23$

$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - Y_{tp}) = 270.874 \cdot MPa$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - h_s - Y_{tp}) = -62.383 \cdot MPa$

$\psi_{sp3.x} := \max(3 + \psi_{sp.x}, 0) = 2.77$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.23$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{\lambda_p - 0.055 \cdot (\psi_{sp3.x})}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$\sigma_{E.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_s^{\,2}} = 180.432 \cdot MPa$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp.x} + 9.78 \cdot \psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98 \cdot (1 - \psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 9.777$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 1.764 \times 10^3 \cdot MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{\,2}} = 6.833 \cdot MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5 \cdot \left[1 + \alpha_{sp} \cdot (\lambda_p - 0.2) + \lambda_p^{\,2}\right] = 0.683$

$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^{\,2} - \lambda_p^{\,2}}}, 1\right) = 0.91$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma} - 1, 0\right), 1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \frac{\left[(\rho_{px.sp} - \chi_{sp})\xi_{sp.x} \cdot (2 - \xi_{sp.x}) \cdots \right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp})\xi_{sp.x} \cdot (2 - \xi_{sp.x}) \cdots \right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp})\xi_{sp.x} \cdot (2 - \xi_{sp.x}) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34$ $\lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5 \cdot \left[1 + \alpha_{p.sp} \cdot (\lambda_p - \lambda_{p0.sp}) + \lambda_p\right] = 0.745$

$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^{\,2} - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot \left(1 - \nu^2\right) \cdot s_{ts}^{\,2}} = 6.833 \cdot MPa$$

$$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 27.331 \cdot MPa$$

Collumn-like behaviour

$$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^{\,2}}{12 \cdot \left(1 - \nu^2\right) \cdot h_s^{\,2}} = 180.432 \cdot MPa$$

Interpolation

$$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) \cdots \right] \\ + \chi_{sp} \end{cases} \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) \cdots \right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \quad = 0.9$$

$$1 \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.748 \quad = 1 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^{\,2}} \text{ if } \lambda_p > 0.748 \end{cases}$$

$$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.557$$

$$\sigma_{E.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^{\,2}}{12 \cdot \left(1 - \nu^2\right) \cdot h_{so}^{\,2}} = 2.479 \times 10^3 \cdot MPa$$

$$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 \quad = 4.822 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{tps.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{tps.x} + 9.78 \cdot \psi_{sp.x}^{\,2} \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{tps.x} = -1 \\ 5.98 \cdot \left(1 - \psi_{tps.x}\right)^2 \text{ if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 \text{ if } \psi_{tps.x} < -3 \end{cases}$$

$$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x} \cdot \sigma_{E.tps.x} = 1.195 \times 10^4 \cdot MPa$$

Collumn-like behaviour

$$\sigma_{cr.c.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^{\,2}}{12 \cdot \left(1 - \nu^2\right) \cdot s_{ts}^{\,2}} = 12.147 \cdot MPa$$

$$\chi_{tps} := \chi_{sp}$$

Interpolation

$$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) \cdots \right] \\ + \chi_{tps} \end{cases} \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) \cdots \right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1$$

$$1 \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp} \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp}}{\frac{\chi_w \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.988$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \ldots = 0.676$$

$$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tps}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 = 0.589$$

EWM - 16mm top plate thickness

This chapter covers the EWM when changing the top plate thickness to 16mm.

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 16mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 896 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.142 \cdot m^2$

$A_c := A_{so} + A_d = 0.076 m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 315.07 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.091 \times 10^{-3} m^4$$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 1.22 \times 10^{-3} m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 4.159 \times 10^{-3} m^4$$

Deck:

$$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 5.282 \times 10^{-3} m^4$$

Bottom flange:

$$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 5.252 \times 10^{-3} m^4$$

Side plate:

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 16 \cdot mm$

$z_2 := t_d + h_s = 405.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.301$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 10.594$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.431$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.09\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 299.07 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 299.07 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 119.628 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 179.442 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 7.178 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.911 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1w} := t_d = 16 \cdot mm$

$z_{2w} := t_d + h_{so} = 156 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.532$

$k_{\sigma.4.2b}(\psi_{so}) = 0.663$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.465$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 21.875 \quad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$z_d := \frac{-t_d}{2} = -8 \times 10^{-3} m$ $\psi_d := \psi_{neg}(z_d, z_d) = 1$

$\delta_{so.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.452$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 1$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 334 \cdot mm$

Areas:

$A_{ex} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 7.286 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 7.056 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot (1 - \nu^2)} = 1.313 \times 10^{6} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 28.647 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.243 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 94.689 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.056 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.36$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 214.934$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot (1 - \nu^2) \cdot b_d^{2}} = 852.521 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.968$

$\lambda_{p.} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.635$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 7.59 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 2.695 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 30.286 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.351 \times 10^{7} \cdot mm^4$

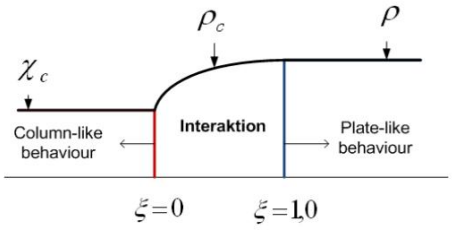
Buckling resistance stiffener

$A_{sl.} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 7.84 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 7.584 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.967$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 893.058 \cdot MPa$

Slenderness ratio

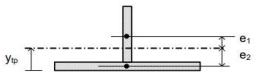
$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.62$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_{.} := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 125.92 \cdot mm$



$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 57.353 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.531$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.804$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.76$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is almost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.76$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -2.184 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -320.028 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 269.913 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.053 \, m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.14 \, m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 320.695 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 2.311 \times 10^{-4} \, m^4$$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.226 \times 10^{-3} \, m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.017 \, m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 7.056 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -6.612 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 589.921 \cdot cm^2$$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.251 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 12.165 \cdot mm$$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 346.215 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 6.114 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206\,mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.398 \times 10^{-3}\,m^4$$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 1.039 \times 10^{-3}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 3.526 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 4.697 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 2.897 \times 10^{-4}\,m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.706 \times 10^{10}\,mm^4$

Load

Load capacity:

$q_M := 123\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_d = 430.5\,\frac{kN}{m}$

$$M_{Ed} := \frac{Q_M \cdot l_{bridge}^{2}}{8} = 1.744 \times 10^{4}\,kN \cdot m$$

$q_V := 134\,\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V \cdot b_d = 469\,\frac{kN}{m}$

$$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.221 \times 10^{3}\,kN$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.049\,m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.75 \times 10^{4}\,kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.997$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$$ Angle of web

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction (two stiffeners)

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$$

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$$a_{stw.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15st of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.455$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\epsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \epsilon\left(t_w\right)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$

SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot \text{kN}$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221 \times 10^3 \cdot \text{kN}$

$\frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$ (Contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

S83 "bärande konstruktioner"

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 338.215 \cdot \text{mm}$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 541.785 \cdot \text{mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left|a_{öf.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y\left(t_b\right)}{\gamma_{M0}} = 9.494 \times 10^3 \cdot \text{kN} \cdot \text{m}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.814\,\text{m}$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59\,\text{m}$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 453.954 \cdot \text{kN}$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{kN}$$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot \text{kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2V_{b.Rd}} = 0.993$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.011 \cdot \text{m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 596.641 \cdot \text{cm}^2$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) \dots + \left[n_{so}\cdot t_{so}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) + A_b\cdot\left(h_{ts}-\frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 242.756\cdot mm$$

$$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.045\cdot m^3$$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 1.581\times 10^4\cdot kN\cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 1.75\times 10^4\cdot kN\cdot m$$

Moment capacity check when $q_M = 123\cdot\frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.997$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 134\cdot\frac{kN}{m^2}$

$$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$$

The Reduced stress method

-16mm top plate thickness

Material

$f_y(t) := \begin{cases} 355\,\text{MPa} & \text{if } t \le 40\,\text{mm} \\ (335\,\text{MPa}) & \text{if } t > 40\,\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\,\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\,\text{GPa}$

Geometry

Bottom plate

$t_b := 16\,\text{mm}$ $b_b := 1000\,\text{mm}$ $A_b := b_b \cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 16\,\text{mm}$ $b_d := 3500\,\text{mm}$ $A_d := b_d \cdot t_d = 0.056\,\text{m}^2$

Side plates

$t_s := 12\,\text{mm}$ $h_s := 389.2\,\text{mm}$ $A_s := h_s \cdot t_s = 4.67\times 10^{-3}\,\text{m}^2$

Webs

$t_w := 12\,\text{mm}$ $h_w := 1340\,\text{mm}$ $A_w := h_w \cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\,\text{mm}$ $h_{w.y} := 482.8\,\text{mm}$

Top stiffeners

$t_{so} := 16\,\text{mm}$ $h_{so} := 140\,\text{mm}$ $A_{so} := h_{so} \cdot t_{so} = 2.24\times 10^{-3}\,\text{m}^2$

$s_{so} := 350\,\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\,\text{mm}$ $h_{su} := 120\,\text{mm}$ $A_{su} := h_{su} \cdot t_{su} = 1.44\times 10^{-3}\,\text{m}^2$

$s_{su} := 440\,\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\,\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.896\,\text{m}$ $s_{ts} := 2\,\text{m}$

$I_{tot} := 0.017067\,\text{m}^4$ From grasshopper

$Y_{tp} := 0.5793\,\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 106\,\frac{\text{kN}}{\text{m}^2}$

$Q := q \cdot b_d = 371\,\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q \cdot L_{bridge}^2}{8} = 1.503\times 10^4\,\text{kN·m}$

$V_{Ed} := \frac{Q \cdot L_{bridge}}{2} = 3.339\times 10^3\,\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}} \cdot \left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 271.774\,\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d \cdot b_d}{n_{so}+2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.572\times 10^{-3}\,\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_{tot} \cdot \min(t_s, t_{so})} = 25.622\,\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - Y_{tp}) = 264.731\,\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 15.458\,\text{MPa}$

$S_{sp} := (h_{ts} - t_d - Y_{tp}) \cdot t_s \cdot \frac{(h_{ts} - t_d - Y_{tp})}{2} + \frac{t_d \cdot b_d}{2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 0.012\,\text{m}^3$
$+ \frac{n_{so}}{2} \cdot h_{so} \cdot t_{so}\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5 V_{Ed} \cdot S_{sp}}{I_{tot} \cdot t_s} = 93.839\,\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts} - t_d - Y_{tp}\right) = 264.731\,\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.856\times 10^{-3}\,\text{m}^3$
$+ \frac{t_d \cdot b_d}{n_{so}+2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed} \cdot S_{tps}}{I_{tot} \cdot t_{so}} = 22.69\,\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 3.3\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT} q = 349.8\,\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.015$

$\alpha_{cr.UNIT} := 339.4\,\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 3.202$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.563$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2 \cdot E \cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 3.966\,\text{MPa}$

$I_{p.tp} := \frac{b_d \cdot t_d^3}{12\cdot(1-\nu^2)} = 1.313\times 10^{-6}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}A_{so}} = 0.029\,\text{m}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + n_{so}\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 94.689$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.36$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 214.934$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp} \cdot \sigma_{E.tp} = 852.521 \cdot MPa$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \frac{3 - \psi_{tp}}{5 - \psi_{tp}} \cdot s_{so} = 0.175m$

$b_{2.sup.tp} := \frac{2}{5 - \psi_{tp}} \cdot s_{so} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35m$

$y_{tp.sl.1.tp} := \frac{t_d \cdot b_{sl.1.tp} \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right)}{t_d \cdot b_{sl.1.tp} + A_{so}} = 0.03m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp} \cdot t_d^3}{12} + b_{sl.1.tp} \cdot t_d \cdot \left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{so} \cdot h_{so}^3}{12} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right)^2$

Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2 \cdot E \cdot I_{sl.1.tp}}{A_{sl.1.tp} \cdot s_{ts}^2} = 893.058 \cdot MPa$ Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.042m$

$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d \cdot t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.056m$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.611$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e.tp} \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.769$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1\right) = 0.773$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right) \cdot \xi_{tp} \cdot \left(2 - \xi_{tp}\right) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right) \cdot \xi_{tp} \cdot \left(2 - \xi_{tp}\right) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right) \cdot \xi_{tp} \cdot \left(2 - \xi_{tp}\right) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.773$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \leq \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \geq 1.08 \end{cases} = 1.2$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})} \cdot z + \sigma_{max}\right] \cdot -1$

$\psi_{sp.x} := \frac{\sigma\left(h_{ts} - t_d - h_s\right)}{\sigma\left(h_{ts} - t_d\right)} = -0.294$

$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}} \cdot \left(h_{ts} - t_d - Y_{tp}\right) = 264.731 \cdot MPa$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}} \cdot \left(h_{ts} - t_d - h_s - Y_{tp}\right) = -77.914 \cdot MPa$

$\psi_{sp3.x} := \max\left(3 + \psi_{sp.x}, 0\right) = 2.706$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.294$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{\lambda_p - 0.055 \cdot (\psi_{sp3.x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$\sigma_{E.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot \left(1 - \nu^2\right) \cdot h_s^2} = 180.432 \cdot MPa$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp.x} + 9.78 \cdot \psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98 \cdot \left(1 - \psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 10.508$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 1.896 \times 10^3 \cdot MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot \left(1 - \nu^2\right) \cdot s_{ts}^2} = 6.833 \cdot MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp} \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.697$

$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1\right) = 0.903$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right) \cdot \xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right) \cdot \xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right) \cdot \xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34$ $\lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp} \cdot (\lambda_p - \lambda_{p0.sp}) + \lambda_p\right] = 0.758$

$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 6.833 \cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 27.331 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 180.432 \cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) \cdots\right] & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{sp} & \\ 1 \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 & \end{cases} = 0.903$

Top plate stiffeners - check

x-direction

$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.748 & = 1 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} \text{ if } \lambda_p > 0.748 & \end{cases}$

$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.534$

$\sigma_{E.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^2}{12 \cdot (1 - \nu^2) \cdot h_{so}^2} = 2.479 \times 10^3 \cdot MPa$

$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 & = 5.296 \\ \frac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \text{ if } 1 > \psi_{sp.x} > 0 & \\ 7.81 \text{ if } \psi_{tps.x} = 0 & \\ 7.81 - 6.29 \cdot \psi_{tps.x} + 9.78 \cdot \psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 & \\ 23.9 \text{ if } \psi_{tps.x} = -1 & \\ 5.98 \cdot \left(1 - \psi_{tps.x}\right)^2 \text{ if } -1 > \psi_{sp.x} \geq -3 & \\ 95.68 \text{ if } \psi_{tps.x} < -3 & \end{cases}$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x} \cdot \sigma_{E.tps.x} = 1.313 \times 10^4 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 12.147 \cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \cdots\right] & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{tps} & \\ 1 \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 & \end{cases} = 1$

Top plate check

$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp} \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp}}{\frac{\chi_w \cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.992$

Side plate check

$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.668$

$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$

Top plate stiffeners check

$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x} \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tps}}{\frac{\chi_w \cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 = 0.565$

EWM - 18mm top plate thickness

This chapter covers the EWM when changing the top plate thickness to 18mm.

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 18mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 898 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.149 \cdot m^2$

$A_c := A_{so} + A_d = 0.083 m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 302.251 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 9.583 \times 10^{-4} m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 1.311 \times 10^{-3} m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 4.482 \times 10^{-3} m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 5.419 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 5.528 \times 10^{-3} m^4$

Side plate:

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 18 \cdot mm$

$z_2 := t_d + h_s = 407.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.369$

$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$

$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$k_{\sigma.4.1}(\psi_s) = 11.466$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.415$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.105\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 284.251 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 284.251 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 113.7 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 170.55 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 6.822 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 2.069 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1w} := t_d = 18 \cdot mm$

$z_{2w} := t_d + h_{so} = 158 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.507$

$k_{\sigma.4.2b}(\psi_{so}) = 0.682$ Higher stress close to support

$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.459$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 19.444 \quad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$z_d := \frac{-t_d}{2} = -9 \times 10^{-3} m$ $\psi_d := \psi_{neg}(z_d, z_d) = 1$

$\delta_{sso} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.402$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 1$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 334 \cdot mm$

Areas:

$A_{sso} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 7.945 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 7.686 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot (1 - \nu^2)} = 1.869 \times 10^{6} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 28.152 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.299 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 69.518 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.063 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.32$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 163.051$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot (1 - \nu^2) \cdot b_d^{2}} = 818.517 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.967$

$\lambda_{p.} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.648$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 8.287 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 2.942 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 29.721 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.414 \times 10^{7} \cdot mm^4$

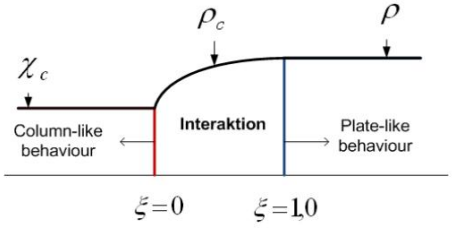
Buckling resistance stiffener

$A_{sl.} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 8.54 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 8.252 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.966$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 858.034 \cdot MPa$

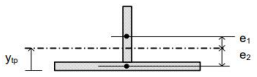
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.632$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_{} := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 123.354 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 59.848 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.534$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.815$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.752$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is almost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.752$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -2.371 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -319.242 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 266.946 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.06\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.146\,m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 308.019 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 2.03 \times 10^{-4}\,m^4$$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.377 \times 10^{-3}\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.018\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 7.686 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -7.141 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 638.077 \cdot cm^2$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.299 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 13.535 \cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 335.107 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 6.395 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206\,mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.264 \times 10^{-3}\,m^4$$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 1.113 \times 10^{-3}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 3.785 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 4.927 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 2.581 \times 10^{-4}\,m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.774 \times 10^{10}\,mm^4$

Load

Load capacity:

$q_M := 132\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_d = 462 \cdot \frac{kN}{m}$

$$M_{Ed} := \frac{Q_M \cdot l_{bridge}^{2}}{8} = 1.871 \times 10^{4} \cdot kN \cdot m$$

$q_V := 134\,\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V \cdot b_d = 469 \cdot \frac{kN}{m}$

$$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.221 \times 10^{3} \cdot kN$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.053 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.879 \times 10^{4} \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.996$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$

$$\alpha_w := \asin\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$$ Angle of web

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$$a_{stw.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15εt of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.455$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon\left(t_w\right)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$

SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot \text{kN}$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221 \times 10^3 \cdot \text{kN}$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.993$ (contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

S83 "bärande konstruktioner"

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 326.107 \cdot \text{mm}$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 554.893 \cdot \text{mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left|a_{öf.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y\left(t_b\right)}{\gamma_{M0}} = 1.011 \times 10^4 \cdot \text{kN} \cdot \text{m}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.898\,\text{m}$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59\,\text{m}$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 502.117 \cdot \text{kN}$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{kN}$$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot \text{kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.993$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.011 \cdot \text{m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 646.481 \cdot \text{cm}^2$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \ldots + \left[n_{so} \cdot t_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 225.57 \cdot mm$$

$$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.05 \cdot m^3$$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.782 \times 10^4 \cdot kN \cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 1.879 \times 10^4 \cdot kN \cdot m$$

Moment capacity check when $q_M = 132 \cdot \frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.996$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The Reduced stress method

-18mm top plate thickness

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 18\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.063\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.898\,\text{m}$ $s_{ts} := 2\text{m}$

$I_{tot} := 0.017682\,\text{m}^4$ From grasshopper

$Y_{tp} := 0.59348\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 112\,\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 392\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.588\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 3.528\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 265.336\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.693\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 28.142\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 257.255\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 16.333\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 0.012\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\,t_s} = 99.629\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 257.255\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.961\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 24.46\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 3.05\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT}\cdot q = 341.6\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.039$

$\alpha_{cr.UNIT} := 343.416\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 3.066$ =

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.582$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases}$ $= 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 5.02\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 1.869\times10^{-6}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\,A_{so}} = 0.028\,\text{m}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 69.518$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.32$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\cdot\left[\left(1+\alpha_{tp}{}^2\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}{}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 163.051$$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 818.517\cdot MPa$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \dfrac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$

$b_{2.sup.tp} := \dfrac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$

$y_{tp.sl.1.tp} := \dfrac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\cdot b_{sl.1.tp}+A_{so}} = 0.03\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \dfrac{b_{sl.1.tp}\cdot t_d{}^3}{12} + b_{sl.1.tp}\cdot t_d\cdot\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \dfrac{t_{so}\cdot h_{so}{}^3}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$ Combined moment of inertia

$\sigma_{cr.sl.tp} := \dfrac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}{}^2} = 858.034\cdot MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\dfrac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.041\,m$

$e_{tp} := \max\left(y_{tp.sl.1.tp}-t_d, t_d+\dfrac{h_{so}}{2}-y_{tp.sl.1.tp}\right) = 0.058\,m$

$\alpha_{e.tp} := 0.49 + \dfrac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.619$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\cdot\left[1+\alpha_{e.tp}\cdot(\lambda_p-0.2)+\lambda_p{}^2\right] = 0.788$ [6.3.1.2]

$\chi_{tp} := \min\left(\dfrac{1}{\phi_{tp}+\sqrt{\phi_{tp}{}^2-\lambda_p{}^2}}, 1\right) = 0.759$

Interpolation

$\xi_{tp} := \min\left(\max\left(\dfrac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}}-1, 0\right), 1\right) = 0$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot(2-\xi_{tp})+\chi_{tp}\right] & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot(2-\xi_{tp})+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot(2-\xi_{tp})+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.759$$

Shear

$\eta := 1.2$ 7.1 (2)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \dfrac{0.83}{\eta} \\ \dfrac{0.83}{\lambda_p} & \text{if } \dfrac{0.83}{\eta} \le \lambda_p < 1.08 \\ \dfrac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\dfrac{-\sigma_{max}}{(Y_{tp})}\cdot z+\sigma_{max}\right]^{-1}$

$\psi_{sp.x} := \dfrac{\sigma(h_{ts}-t_d-h_s)}{\sigma(h_{ts}-t_d)} = -0.358$

$\sigma_{x.top} := \dfrac{M_{Ed}}{I_{tot}}\cdot(h_{ts}-t_d-Y_{tp}) = 257.255\cdot MPa$

$\sigma_{x.bot} := \dfrac{M_{Ed}}{I_{tot}}\cdot(h_{ts}-t_d-h_s-Y_{tp}) = -92.192\cdot MPa$

$\psi_{sp3.x} := \max(3+\psi_{sp.x}, 0) = 2.642$ $\psi_{sp1} := \dfrac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.358$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p-0.055\cdot(\psi_{sp3.x})}{\lambda_p{}^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$\sigma_{E.sp.x} := \dfrac{\pi^2\cdot E\cdot t_s{}^2}{12\cdot(1-\nu^2)\cdot h_s{}^2} = 180.432\cdot MPa$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \dfrac{8.2}{(1.05+\psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81-6.29\cdot\psi_{sp.x}+9.78\cdot\psi_{sp.x}{}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot(1-\psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 11.32$$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 2.043\times 10^3\cdot MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \dfrac{\pi^2\cdot E\cdot t_s{}^2}{12\cdot(1-\nu^2)\cdot s_{ts}{}^2} = 6.833\cdot MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\cdot\left[1+\alpha_{sp}\cdot(\lambda_p-0.2)+\lambda_p{}^2\right] = 0.71$

$\chi_{sp} := \min\left(\dfrac{1}{\phi_{sp}+\sqrt{\phi_{sp}{}^2-\lambda_p{}^2}}, 1\right) = 0.897$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\dfrac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}}-1, 0\right), 1\right) = 1$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\cdot\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.771$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1 - \nu^2\right)\cdot s_{ts}^2} = 6.833\cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 27.331\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1 - \nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.511$

$\sigma_{E.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1 - \nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 & \text{if } \psi_{tps.x} = 1 \\ \frac{8.2}{\left(1.05 + \psi_{tps.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{tps.x} < -3 \end{cases} = 5.849$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.45\times 10^4\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1 - \nu^2\right)\cdot s_{ts}^2} = 12.147\cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots + \chi_{tps}\right] & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.984$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.655$$

$$+ 3\cdot\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 = 0.535$$

EWM - 20mm top plate thickness

This chapter covers the EWM when changing the top plate thickness to 20mm.

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 20mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 900 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.156 \cdot m^2$

$A_c := A_{so} + A_d = 0.09\,m^2$

Change of center of gravity, z_{tp}

(From top to centre of gravity)

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 290.669 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 8.447 \times 10^{-4}\,m^4$$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 1.399 \times 10^{-3}\,m^4 + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 4.79 \times 10^{-3}\,m^4$$

Deck:

$$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 5.517 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 5.786 \times 10^{-3}\,m^4$$

Side plate:

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 20 \cdot mm$

$z_2 := t_d + h_s = 409.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.438$

$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$

$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$k_{\sigma.4.1}(\psi_s) = 12.44$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.398$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.119\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 270.669 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 270.669 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 108.268 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 162.402 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 6.496 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 2.245 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$z_{1w} := t_d = 20 \cdot mm$

$z_{2w} := t_d + h_{so} = 160 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.483$

$k_{\sigma.4.2b}(\psi_{so}) = 0.703$ Higher stress close to support

$Class4_w := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.452$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^4 \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 17.5 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$z_d := \frac{-t_d}{2} = -0.01\,m$ $\psi_d := \psi_{neg}(z_d, z_d) = 1$

$\delta_{so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.361$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 1$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 334 \cdot mm$

Areas:

$A_{ex} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 8.604 \times 10^4 \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 8.316 \times 10^4 \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 2.564 \times 10^6 \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 27.888 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.354 \times 10^8 \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 52.82 \qquad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02\,m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.07\,m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.288$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 127.397$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^2}{12 \cdot (1 - \nu^2) \cdot b_d^2} = 789.548 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.967$

$\lambda_{p.} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.659$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 8.984 \times 10^4 \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 3.189 \times 10^4 \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 29.394 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^3}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.475 \times 10^7 \cdot mm^4$

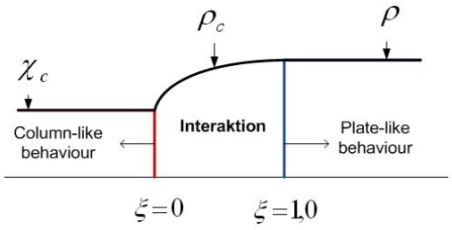
Buckling resistance stiffener

$A_{sl.} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 9.24 \times 10^3 \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 8.92 \times 10^3 \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.965$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 827.286 \cdot MPa$

Slenderness ratio

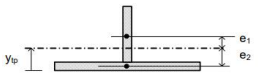
$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.644$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_{} := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 121.068 \cdot mm$



$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 62.112 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.536$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.826$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.744$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is almost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.744$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -2.555 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -318.251 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 264.165 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.067 \, m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.153 \, m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 296.536 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 1.806 \times 10^{-4} \, m^4$$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.487 \times 10^{-3} \, m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.019 \, m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

5.2.1 Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 8.316 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -7.67 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 685.617 \cdot cm^2$

A_{su} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.347 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 14.883 \cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 325.009 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 6.631 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206\,mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.146 \times 10^{-3}\,m^4$$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 1.184 \times 10^{-3}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4$ from FEM

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 4.033 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 5.144 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 2.318 \times 10^{-4}\,m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.837 \times 10^{10}\,mm^4$

Load

Load capacity:

$q_M := 141\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_d = 493.5\,\frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^{2}}{8} = 1.999 \times 10^{4}\,kN \cdot m$

$q_V := 134\,\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V \cdot b_d = 469\,\frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.221 \times 10^{3}\,kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.057\,m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 2.007 \times 10^{4}\,kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.996$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$

$\alpha_w := \asin\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$$a_{stw.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15st of the web z-direction

$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon\left(t_w\right)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$

SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot \text{kN}$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.221 \times 10^3 \cdot \text{kN}$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.993$ (contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

S83 "bärande konstruktioner"

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 315.009 \cdot \text{mm}$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 566.991 \cdot \text{mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left|a_{öf.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} \ldots = 1.069 \times 10^4 \cdot \text{kN} \cdot \text{m}$
$+ \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y\left(t_b\right)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.991 \text{ m}$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59 \text{ m}$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 546.346 \cdot \text{kN}$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{kN}$$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot \text{kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.993$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.012 \cdot \text{m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 695.6 \cdot \text{cm}^2$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) \dots + \left[n_{so}\cdot t_{so}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) + A_b\cdot\left(h_{ts}-\frac{t_b}{2}\right)\right]}{A_{eff.f.tot}} = 211.151\cdot mm$$

$$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.056\cdot m^3$$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 1.98\times 10^4\cdot kN\cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 2.007\times 10^4\cdot kN\cdot m$$

Moment capacity check when $q_M = 141\cdot\frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.996$$

EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 134\cdot\frac{kN}{m^2}$

$$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.993$$

The Reduced stress method

-20mm top plate thickness t_d

Material

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ (335MPa) & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} := 1$

$E := 210GPa$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot GPa$

Geometry

Bottom plate

$t_b := 16mm$ $b_b := 1000mm$ $A_b := b_b\cdot t_b = 0.016\,m^2$

Top plate

$t_d := 20mm$ $b_d := 3500mm$ $A_d := b_d\cdot t_d = 0.07\,m^2$

Side plates

$t_s := 12mm$ $h_s := 389.2mm$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,m^2$

Webs

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w\cdot t_w = 0.016\,m^2$

$h_{w.x} := 1250mm$ $h_{w.y} := 482.8mm$

Top stiffeners

$t_{so} := 16mm$ $h_{so} := 140mm$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,m^2$

$s_{so} := 350mm$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12mm$ $h_{su} := 120mm$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,m^2$

$s_{su} := 440mm$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18m$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.9\,m$ $s_{ts} := 2m$ $s_{ts0} := 0.5m$

$I_{tot} := 0.018242m^4$ From grasshopper

$Y_{tp} := 0.606392m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 119\frac{kN}{m^2}$

$Q := q\cdot b_d = 416.5\cdot\frac{kN}{m}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.687\times10^4\cdot kN\cdot m$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 3.748\times10^3\cdot kN$

$M_{Ed.end} := V_{Ed}\cdot s_{ts} - \frac{Q\cdot(s_{ts})^2}{2} = 6.664\times10^3\cdot kN\cdot m$

$V_{Ed.mid} := V_{Ed} - Q\cdot 8m = 416.5\cdot kN$

Mid top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 262.25\cdot MPa$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.805\times10^{-3}\cdot m^3$

$\tau_{Ed.tp} := \frac{V_{Ed.mid}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 3.434\cdot MPa$

Mid side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 253.003\cdot MPa$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 17.354\cdot MPa$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 0.012\cdot m^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed.mid}\cdot S_{sp}}{I_{tot}\,t_s} = 11.823\cdot MPa$

Mid top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 253.003\cdot MPa$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 2.06\times10^{-3}\cdot m^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed.mid}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 2.939\cdot MPa$

Webs

$\sigma_{x.Ed.w} := \frac{M_{Ed.end}}{I_{tot}}\cdot\left(h_{ts} - t_d - h_s - Y_{tp}\right) = -42.227\cdot MPa$

$Y_{tp.su1} := 0.076773m$ From grasshopper $\sigma_{x.Ed.ww} := \frac{M_{Ed.end}}{I_{tot}}\cdot\left(t_b - Y_{tp}\right) = -215.677\cdot MPa$

$Y_{tp.su2} := 0.235306m$

$Y_{tp.su.3} := 0.393836m$

$Y_{tp.w} := 0.249383m$

$S_b := \frac{A_b}{2}\cdot\left(Y_{tp} - \frac{t_b}{2}\right) + A_{su}\cdot\left(Y_{tp} - Y_{tp.su1}\right) + A_{su}\cdot\left(Y_{tp} - Y_{tp.su2}\right) \ldots = 0.012\cdot m^3$
$+ A_{su}\cdot\left(Y_{tp} - Y_{tp.su.3}\right) + A_w\cdot\left(Y_{tp} - Y_{tp.w}\right)$

$\tau_{Ed.w} := \frac{0.5V_{Ed}\cdot S_b}{I_{tot}\cdot t_w} = 103.865\cdot MPa$

$\sigma_{z.Ed.w} := \frac{Q}{2\cdot t_w} = 17.354\cdot MPa$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 2.84\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT}\cdot q = 337.96\cdot MPa$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.05$

$\alpha_{cr.UNIT} := 347.02\cdot\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.916$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.6$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour [A.1]

$$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^{\,2}}{12\cdot(1-\nu^2)\cdot b_d^{\,2}} = 6.198\cdot MPa$$

$$I_{p.tp} := \frac{b_d\cdot t_d^{\,3}}{12\cdot(1-\nu^2)} = 2.564\times 10^{-6}\,m^4$$

$$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{A_d + n_{so}A_{so}} = 0.028\,m$$

$$I_{sl.tp} := \frac{b_d\cdot t_d^{\,3}}{12} + A_d\cdot\left(y_{tp.sl.tp}-\frac{t_d}{2}\right)^2 + n_{so}\left[\frac{t_{so}\cdot h_{so}^{\,3}}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.tp}\right)^2\right]$$

$$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 52.82$$

$$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.288$$

$$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\cdot\left[\left(1+\alpha_{tp}^{\,2}\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}^{\,2}\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 127.397$$

$$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 789.548\cdot MPa$$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

$$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$$

$$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\cdot b_{sl.1.tp} + A_{so}} = 0.029\,m$$

combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^{\,3}}{12} + b_{sl.1.tp}\cdot t_d\cdot\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \frac{t_{so}\cdot h_{so}^{\,3}}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}^{\,2}} = 827.286\cdot MPa$$

critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.04\,m$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d, t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.061\,m$$

$$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{to}}} = 0.627$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5\cdot\left[1+\alpha_{e.tp}\cdot(\lambda_p-0.2)+\lambda_p^{\,2}\right] = 0.805 \qquad [6.3.1.2]$$

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}^{\,2}-\lambda_p^{\,2}}}, 1\right) = 0.745$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[(\rho_{px.tp}-\chi_{tp})\xi_{tp}\cdot(2-\xi_{tp})+\chi_{tp}\right] & \text{if } \dfrac{\left[(\rho_{px.tp}-\chi_{tp})\xi_{tp}\cdot(2-\xi_{tp})+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[(\rho_{px.tp}-\chi_{tp})\xi_{tp}\cdot(2-\xi_{tp})+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.745$$

Shear

$$\eta := 1.2 \qquad 7.1\ (2)$$

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \dfrac{0.83}{\eta} \\ \dfrac{0.83}{\lambda_p} & \text{if } \dfrac{0.83}{\eta} \le \lambda_p < 1.08 \\ \dfrac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})}\cdot z + \sigma_{max}\right]\cdot -1$$

$$\psi_{sp.x} := \frac{\sigma(h_{ts}-t_d-h_s)}{\sigma(h_{ts}-t_d)} = -0.422$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts}-t_d-Y_{tp}) = 253.003\cdot MPa$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts}-t_d-h_s-Y_{tp}) = -106.887\cdot MPa$$

$$\psi_{sp3.x} := \max(3+\psi_{sp.x}, 0) = 2.578 \qquad \psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.422$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p - 0.055\cdot(\psi_{sp3.x})}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\cdot(1-\nu^2)\cdot h_s^{\,2}} = 180.432\cdot MPa$$

$$k_{\sigma.sp.x} := \begin{cases} 1 \text{ if } \psi_{sp.x} = 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp.x} + 9.78 \cdot \psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{sp.x} = -1 \\ 5.98 \cdot (1 - \psi_{sp.x})^2 \text{ if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 \text{ if } \psi_{sp.x} < -3 \end{cases} = 12.213$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 2.204 \times 10^3 \cdot MPa$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 6.833 \cdot MPa$$

$$\alpha_{sp} := 0.21$$

$$\phi_{sp} := 0.5 \cdot \left[1 + \alpha_{sp} \cdot (\lambda_p - 0.2) + \lambda_p^2\right] = 0.722$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1\right) = 0.89$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right) \xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) \cdots\right] \\ + \chi_{sp} \end{cases} \text{ if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right) \xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \quad = 1$$
$$1 \text{ if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right) \xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1$$

Z-direction

$$\psi_{sp.z} := 1$$

$$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$$

$$\phi_{p.sp} := 0.5 \cdot \left[1 + \alpha_{p.sp} \cdot \left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.783$$

$$\rho_{pz.sp} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} \text{ if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$$k_{\sigma.p.sp.z} := 4$$

$$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 6.833 \cdot MPa$$

$$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 27.331 \cdot MPa$$

Collumn-like behaviour

$$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 180.432 \cdot MPa$$

Interpolation

$$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right) \xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) \cdots\right] \\ + \chi_{sp} \end{cases} \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right) \xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \quad = 0.8$$
$$1 \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right) \xi_{sp.z} \cdot \left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} \text{ if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.488$$

$$\sigma_{E.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^2}{12 \cdot (1 - \nu^2) \cdot h_{so}^2} = 2.479 \times 10^3 \cdot MPa$$

$$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 \\ \frac{8.2}{(1.05 + \psi_{tps.x})} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{tps.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{tps.x} + 9.78 \cdot \psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{tps.x} = -1 \\ 5.98 \cdot (1 - \psi_{tps.x})^2 \text{ if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 \text{ if } \psi_{tps.x} < -3 \end{cases} = 6.484$$

$$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x} \cdot \sigma_{E.tps.x} = 1.607 \times 10^4 \cdot MPa$$

Collumn-like behaviour

$$\sigma_{cr.c.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 12.147 \cdot MPa$$

$$\chi_{tps} := \chi_{sp}$$

Interpolation

$$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right) \xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) \cdots\right] \\ + \chi_{tps} \end{cases} \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right) \xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) \cdots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1$$
$$1 \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right) \xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1$$

Web plates check

x-direction

the whole web is in tension in x-direction

$$\rho_{c.w.x} := 1$$

y-direction

$$\psi_{w.z} := 1$$

$$\alpha_{p.w} := 0.34 \qquad \lambda_{p0.w} := 0.7$$

$$\phi_{p.w} := 0.5 \cdot \left[1 + \alpha_{p.sp} \cdot \left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.783$$

$$\rho_{pz.w} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} \text{ if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.w.z} := 4$

$\sigma_{E.w.z} := \frac{\pi^2 \cdot E \cdot t_w^{\,2}}{12 \cdot (1 - \nu^2) \cdot s_{ts0}^{\,2}} = 109.325 \cdot MPa$

$\sigma_{cr.p.w.z} := k_{\sigma.p.w.z} \cdot \sigma_{E.w.z} = 437.299 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.w.z} := \frac{\pi^2 \cdot E \cdot t_w^{\,2}}{12 \cdot (1 - \nu^2) \cdot h_w^{\,2}} = 15.221 \cdot MPa$

$\chi_{wp} := \chi_{sp}$

Interpolation

$\xi_{w.z} := \min\left(\max\left(\frac{\sigma_{cr.p.w.z}}{\sigma_{cr.c.w.z}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.w.z} := \begin{cases} \left[(\rho_{pz.w} - \chi_{wp}) \cdot \xi_{w.z} \cdot (2 - \xi_{w.z}) \cdots \right] \\ + \chi_w \end{cases} \text{ if } \frac{\left[(\rho_{pz.w} - \chi_{wp}) \cdot \xi_{w.z} \cdot (2 - \xi_{w.z}) \cdots \right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \quad = 1$$
$$1 \text{ if } \frac{\left[(\rho_{pz.w} - \chi_{wp}) \cdot \xi_{w.z} \cdot (2 - \xi_{w.z}) + \chi_{wp} \right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp} \cdot f_y(t_d)}{\gamma_{M1}}} \right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp}}{\frac{\chi_w \cdot f_y(t_d)}{\gamma_{M1}}} \right)^2 = 0.984$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y(t_s)}{\gamma_{M1}}} \right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y(t_s)}{\gamma_{M1}}} \right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x} \cdot f_y(t_s)}{\gamma_{M1}}} \right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z} \cdot f_y(t_s)}{\gamma_{M1}}} \right) \cdots = 0.474$$
$$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y(t_s)}{\gamma_{M1}}} \right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x} \cdot f_y(t_s)}{\gamma_{M1}}} \right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tps}}{\frac{\chi_w \cdot f_y(t_s)}{\gamma_{M1}}} \right)^2 = 0.508$$

Web plate check

$$\left(\frac{\sigma_{x.Ed.w}}{\frac{\rho_{c.w.x} \cdot f_y(t_s)}{\gamma_{M1}}} \right)^2 + \left(\frac{\sigma_{z.Ed.w}}{\frac{\rho_{c.w.z} \cdot f_y(t_s)}{\gamma_{M1}}} \right)^2 - \left(\frac{\sigma_{x.Ed.w}}{\frac{\rho_{c.w.x} \cdot f_y(t_s)}{\gamma_{M1}}} \right) \cdot \left(\frac{\sigma_{z.Ed.w}}{\frac{\rho_{c.w.z} \cdot f_y(t_s)}{\gamma_{M1}}} \right) + 3 \cdot \left(\frac{\tau_{Ed.w}}{\frac{\chi_w \cdot f_y(t_s)}{\gamma_{M1}}} \right)^2 = 0.58$$

C.3 Changed top plate stiffener thickness

CXCV	4 mm top plate stiffener EWM calculation.
CXCVIII	4 mm top plate stiffener RSM calculation.
CC	6 mm top plate stiffener EWM calculation.
CCIII	6 mm top plate stiffener RSM calculation.
CCV	8 mm top plate stiffener EWM calculation.
CCVIII	8 mm top plate stiffener RSM calculation.
CCX	10 mm top plate stiffener EWM calculation.
CCXIII	10 mm top plate stiffener RSM calculation.
CCXV	12 mm top plate stiffener EWM calculation.
CCXVIII	12 mm top plate stiffener RSM calculation.
CCXX	14 mm top plate stiffener EWM calculation.
CCXXIII	14 mm top plate stiffener RSM calculation.
CCXXV	18 mm top plate stiffener EWM calculation.
CCXXVIII	18 mm top plate stiffener RSM calculation.

EWM - Changed top plate stiffener thickness

This chapter covers the EWM when changing the top plate stiffener thickness to 4 mm

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s,so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d,0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s,so}} = 350\,mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 4mm$ $s_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $s_{su} := 5mm$

$h_{su,y} := h_{su}\cdot\cos\left(21.12\cdot\frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864\,mm$

Poissons ratio: $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d\cdot b_d$ **parameters:** $A_s := 2\cdot t_s\cdot h_s$ $A_w := 2\cdot t_w\cdot h_w$ $A_b := t_b\cdot b_b$

$A_{so} := n_{so}\cdot t_{so}\cdot h_{so}$ $A_{su} := n_{su}\cdot t_{su}\cdot h_{su}$

$E := 210GPa$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.106\,m^2$ -1 tab. 3.1

$A_c := A_{tot} + A_d = 0.04\,m^2$ ise

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{\begin{aligned} &A_d\frac{t_d}{2} + A_{so}\left(t_d + \frac{h_{so}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ &+ A_b\left(t_d + h_s + h_{wy}\right) + A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right] \end{aligned}}{A_{tot}} = 403.409\,mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\,mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so}\cdot\frac{t_{so}\cdot h_{so}^3}{12} + n_{so}\cdot t_{so}\cdot h_{so}\left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 5.354\times10^{-4}\,m^4$

$I_{y.st.su.1} := 6\cdot1.506\cdot10^{-5}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2\cdot n_{su}\cdot h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 7.239\times10^{-4}\,m^4$$
$$+ 2\cdot n_{su}\cdot h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2\cdot n_{su}\cdot h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2\cdot0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2\cdot h_w\left[h_{ts} - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 2.433\times10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d\cdot t_d^3}{12} + t_d\cdot b_d\left(\frac{t_d}{2} - z_{tp}\right)^2 = 5.556\times10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b\cdot t_b^3}{12} + b_b\cdot b_b\left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 3.665\times10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\cdot\frac{t_s\cdot h_s^3}{12} + 2\cdot t_s\cdot h_s\left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 4.871\times10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.013\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}}\cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}}\cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1-\psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(b_w, h, t) := \frac{\frac{b_w}{t}}{28.4\cdot\varepsilon(t)\cdot\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\cdot\max(3+\psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2) Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.011$

$$Condition := \begin{cases} 62\cdot\varepsilon(t_s)\cdot(1-\psi_s)\cdot\sqrt{-\psi_s} & \text{if } \psi_s \le -1 \\ \frac{42\cdot\varepsilon(t_s)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 7.731$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(s_{c.so}, k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.505$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = -4.209\times10^{-3}\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 393.409\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 393.409\,mm$

$h_{s.eff.c1} := 0.4\,h_{s.eff.c} = 157.363\,mm$

$h_{s.eff.c2} := 0.6\,h_{s.eff.c} = 236.045\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\,mm$

$A_{s.eff.c} := 2\cdot h_{s.eff.c}\cdot t_s = 9.442\times10^{-3}\,mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t})\cdot t_s = 9.341\times10^{-3}\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := \kappa_{\sigma.4.1}(\psi_s)\cdot\frac{E\cdot\pi^2\cdot t_s^2}{12\left(1-\nu^2\right)\cdot h_s^2} = 1.395\times10^3\,MPa$ EC1993-1-5 eq. A.1

Stiffeners

$t_{so} := t_d = 10\,mm$

$h_{sw} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{sog}(z_2)}{\sigma_{sog}(z_1)} = 0.644$

$k_{\sigma.4.2}(\psi_{so}) = 0.587$ Higher stress close to support

$Class4_{so} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \epsilon \cdot (k_{so}) \cdot \sqrt{k_{\sigma.4.2}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2}(\psi_{so}), b_{so}, t_{so}) = 1.976$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_{so}) = 0.458$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so} \cdot b_{so} = 64.095\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot b_{so.eff} = 2.307 \times 10^{-3}\ m^2$

Deck

$\frac{\lambda_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{-} := \begin{cases} \text{"Class4"} & \text{if } \frac{\lambda_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\ m \qquad \psi_d := \psi_{sog}(z_d, z_d) = 1$

$k_{\sigma.sw} := k_{\sigma.4.1}(\psi_d) = 4$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.749$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.943$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 326.339\ mm$

Areas:

$A_{sw} := n_{so} \cdot s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (s_{so.eff} + t_d) = 3.417 \times 10^{-6}\ m^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (s_{so.eff} + t_d) = 3.204 \times 10^{-6}\ m^2$

More than three longitudinal stiffeners -> $k_{\sigma p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5}\ mm^4$

$y_{gp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot s_{so} + b_d \cdot t_d} = 14.441\ mm$

$I_d := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{gp.sl}\right)^2\right] + \left[\frac{n_{so} \cdot t_{so} \cdot s_{so}^3}{12} + n_{so} \cdot s_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{gp.sl}\right)^2\right] = 3.331 \times 10^{-7}\ mm^4$

$\gamma := \frac{I_d}{I_p} = 103.912 \qquad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 5.04 \times 10^{-3}\ m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035\ m^2$

$\beta_A := \frac{A_{sl}}{A_p} = 0.144$

$\lambda_{sw} := \begin{cases} \sqrt{\frac{A_{c.eff.loc} \cdot f_y(t_d)}{\sigma_{cr.sp}}} = 0.876 & \text{if } \alpha \le \sqrt{\gamma} \\ \frac{2}{\alpha \cdot \sqrt{\gamma}} \cdot \left(1 - \frac{1}{2 \cdot \alpha \cdot \sqrt{\gamma}}\right) & \text{otherwise} \end{cases} = 280.207$

$\rho := \rho_{4.2}(\lambda_{sw}, \psi) = 0.855$ Plate like behaviour

Critical buckling stress, ortrotropic plate:

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_{sw}, \psi) \cdot A_{c.eff.loc} + n_{so} \cdot s_{so.eff} \cdot t_d = 3.066 \times 10^{-6}\ m^2$ 3-1-5 eqA.1

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.088 \times 10^{3}\ kN$

Column like behaviour

$y_{gp.sl.c} := \frac{n_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot h_{so} \cdot n_{so} + n_{so} \cdot t_d} = 15.345\ mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{gp.sl.c}\right)^2\right] + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{gp.sl.c}\right)^2\right] = 3.659 \times 10^{-5}\ mm^4$

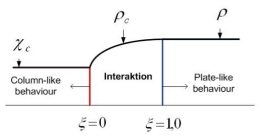
Buckling resistance stiffener

$A_{sw} = t_{so} \cdot h_{so} + n_{so} \cdot t_d = 4.06 \times 10^{-3}\ mm^2$

$A_{sl.eff} := t_{so} \cdot s_{so.eff} + n_{so.eff} \cdot t_d = 3.52 \times 10^{-3}\ mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.867$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot s^2} = 467.022\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sw})}{\sigma_{cr.sl}}} = 0.812$

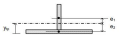
Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_y = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 90.572\ mm$

$e_c := \min\left[\left(y_{gp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{gp.sl}\right)\right] = 65.559\ mm$



$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e_c}} = 0.555$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.999$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.632$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (columnplate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.632$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{sog}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{sog}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.19 \times 10^{-6}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -345.825\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} = 224.387\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EI OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.033\ m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.101\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{s_{so.eff}}{2}\right) + \\ &+ 2 \cdot \left[t_s \cdot b_{s.eff.c1} \cdot \left(t_d + \frac{b_{s.eff.c1}}{2}\right) + \right. \\ &\left. + t_s \cdot b_{s.eff.c2} \cdot \left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2}\right) + t_s \cdot b_{s.t} \cdot \left(t_d + b_s - \frac{b_{s.t}}{2}\right)\right] + \\ &+ A_w \cdot \left(t_d + b_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + b_s + \frac{2}{3}(h_{wy} - s_{so.b}) - \frac{h_{su.y}}{2}\right] + \\ &+ A_b \cdot \left(h_w - \frac{s_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 420.611\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot b_{s.eff.c1}^3}{12} + t_s \cdot b_{s.eff.c1} \cdot \left(t_d + \frac{b_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \frac{t_s \cdot b_{s.eff.c2}^3}{12} + t_s \cdot b_{s.eff.c2} \cdot \left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \frac{t_s \cdot b_{s.t}^3}{12} + t_s \cdot b_{s.t} \cdot \left(t_d + b_s - \frac{b_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 5.538 \times 10^{-4}\ m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.637 \times 10^{-3}\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.014\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{bf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 3.204 \times 10^{-6}\ m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -2.634 \times 10^{-3}\ m^2$

$A_{sl.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 896.546\ cm^2$

$K_{A.2.1} = \text{"EI OK"} \quad \frac{A_{eff.tot}}{2} = 235.138\ cm^2$

According to 4.5.3.(4)Stiffeners type 4, 5 and 6 reduction of the compressive area can be reduced even if the limit of one third is exceeded. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{s.eff} := \begin{cases} \text{Lower stiffener in tension -> no reduction} & \\ \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EI OK"} \\ t_d & \end{cases} = 6.321\ mm$

$\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} - \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot b_{s.eff.c1} \cdot \left(t_d + \frac{b_{s.eff.c1}}{2}\right) - \\ &+ t_s \cdot b_{s.eff.c2} \cdot \left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2}\right) + t_s \cdot b_{s.t} \cdot \left(t_d + b_s - \frac{b_{s.t}}{2}\right) - \\ &+ A_w \cdot \left(t_d + b_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + b_s + \frac{2}{3}(h_{wy} - s_{so.b}) - \frac{h_{su.y}}{2}\right] - \\ &+ A_b \cdot \left(h_w - \frac{s_b}{2}\right) \end{aligned}$

$z_{tp.eff.neg} := \frac{}{A_{eff.tot}} = 463.181\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 6.851 \times 10^{-3}\ m^4$

$\beta_{web} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{so.b} = 7.206\ mm$

Stiffeners overlunder:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot b_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot b_{so.eff} \cdot \left(t_d + \frac{b_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 4.1 \times 10^{-4}\ m^4$

$I_{y.st.su.1.fin} := 6.1.506 \cdot 10^{-6}\ m^4$ from FEM

$\begin{aligned} I_{y.st.su.fin} := {} & I_{y.st.su.1.fin} - \\ & + 2 \cdot s_{su} \cdot b_{su} \cdot \left[t_d + b_s + \frac{1}{3}(h_{wy} - s_{so.by}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 - \\ & + 2 \cdot s_{su} \cdot b_{su} \cdot \left[t_d + b_s + \frac{2}{3}(h_{wy} - s_{so.by}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 - \\ & + 2 \cdot s_{su} \cdot b_{su} \cdot \left[t_d + b_s + (h_{wy} - s_{so.by}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \end{aligned} = 4.895 \times 10^{-4}\ m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\ m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[h_w - \frac{s_b}{2} - \frac{1}{2}(h_{wy}) - z_{tp.eff.neg}\right]^2 = 1.636 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot s_b^3}{12} + t_b \cdot b_b \cdot \left(h_w - z_{tp.eff.neg} - \frac{s_b}{2}\right)^2 = 2.807 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot b_{s.eff.c1}^3}{12} + t_s \cdot b_{s.eff.c1} \cdot \left(t_d + \frac{b_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \frac{t_s \cdot b_{s.eff.c2}^3}{12} + t_s \cdot b_{s.eff.c2} \cdot \left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \frac{t_s \cdot b_{s.t}^3}{12} + t_s \cdot b_{s.t} \cdot \left(t_d + b_s - \frac{b_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 7.425 \times 10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.294 \times 10^{-10}\ mm^4$

Load

$q_M := 69 \frac{kN}{m^2}$

$Q_M := q_M \cdot b_{sl} = 241.5 \frac{kN}{m} \qquad V_{Ed} := \frac{Q_M \cdot b_{bridge}}{2} = 2.174 \times 10^{3}\ kN$

$M_{Ed} := \frac{Q_M \cdot b_{bridge}^2}{8} = 9.781 \times 10^{3}\ kN \cdot m$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.028\ m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 9.915 \times 10^{3}\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.986$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeners

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot t_{sw} + h_w = 0.293\,m$

$\alpha_w := \min\left(\frac{h_{web}}{h_w}, \frac{180}{\pi}\right) = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_b}{2} + s_{wab}\cdot\cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right)\cdot h_{sw}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{sw}\cdot\cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{sw}\cdot\cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2\cdot t_{sw}\cdot h_{sw}\cdot\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18\times 10^{-3}\,m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$\overline{a_{web.1}} := \frac{h_b}{2} + s_{wab}\cdot\cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} = 0.525\,m$

$\overline{a_{web.2}} := a_{st.z.1} + s_{sw}\cdot\cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$

$\overline{a_{web.3}} := a_{st.z.2} + s_{sw}\cdot\cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$

Moment of inirtia of 15st of the web z-direction

$I_{ww.z} := 2\cdot h_{web}\cdot t_w^3\cdot\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$

$I_{w.z.tot} := I_{st.z} + I_{ww.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00\left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34\left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.491$ SS-EN 1993-1-5 5.3 (5) ANM. A3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4\cdot t_w\cdot\varepsilon\cdot\sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\overline{aa} := \frac{a}{h_w} = 1.493$

SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[\left(\frac{h_w}{a}\right)\cdot\sqrt[4]{\frac{I_{st.z.tot}}{\left[\min\left(t_f, t_w\right)^3\cdot h_w\right]}}, \frac{2.1}{\min\left(t_f, t_w\right)}\cdot\sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782\times 10^3$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18\cdot\frac{I_{st.z.tot}}{\min\left(t_f, t_w\right)^3\cdot h_{web}} + 2.2\cdot\sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_f, t_w\right)^3\cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635\times 10^3$

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \\ \text{"Reduce"} & \text{otherwise} \end{cases} = 1.2$ eck has to be performed according to SS-EN 1993-1-5 5.1, stiff and stiffener.

$\frac{h_w}{t_w} \leq 31\cdot\frac{\varepsilon\cdot\sqrt{k_\tau}}{\eta} = \text{"Reduce"}$

Kapacity of the web

$\frac{h_w}{t_w} = 1.00 \cdot$ Shear buckling check needed, EC1003-1-5 Ch. 5.1(2)

$\frac{h_w}{t_w} = 1.00 \cdot \frac{h_w}{37.4\cdot t_w\cdot\varepsilon\cdot\sqrt{k_\tau}} = 1.2$ Satisfy class 3

$\chi_w = 1.00 \cdot \left(\frac{h_w}{37.4\cdot t_w\cdot\varepsilon\cdot\sqrt{k_\tau}} = \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6) SS3 Tabande kontrollrutiner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w\cdot f_y\left(t_f\right)\cdot h_{web}\cdot t_w}{\sqrt{3}\cdot\gamma_{M1}\cdot\gamma_n} = 2.125\times 10^3\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.174\times 10^3\,kN$

$\frac{V_{Ed}}{2\cdot V_{bw.Rd}} = 0.511$ Contribution from two web

$V_{b.Rd} \leq \frac{\eta\cdot f_y\left(t_f\right)\cdot h_{web}\cdot t_w}{\sqrt{3}\cdot\gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{pf.eff.neg} := a_{tp.eff.neg} - \frac{t_d}{2} = 458.181\,mm$

$a_{sf.eff.neg} := h_{tk} - \frac{t_b}{2} - a_{tp.eff.neg} = 418.819\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{pf.eff.neg}\cdot\left|a_{pf.eff.neg}\right|\cdot f_y\left(t_d\right)}{\gamma_{M0}} + \frac{A_{sf.eff.neg}\cdot\left|a_{sf.eff.neg}\right|\cdot f_y\left(t_b\right)}{\gamma_{M0}} = 7.687\times 10^3\,kN\cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{pf} := a\cdot\left[0.25 + \frac{1.6\cdot b_d\cdot t_d^2\cdot f_y\left(t_d\right)}{t_f\cdot\left(b_w + b_{wy}\right)^2\cdot f_y\left(t_f\right)}\right] = 0.623\,m$

$c_{sf} := a\cdot\left[0.25 + \frac{1.6\cdot b_b\cdot t_b^2\cdot f_y\left(t_b\right)}{t_f\cdot\left(b_w + b_{wy}\right)^2\cdot f_y\left(t_f\right)}\right] = 0.59\,m$

$V_{bf.Rd.cored} := \frac{b_d\cdot t_d^2\cdot f_y\left(t_d\right)}{c_{pf}\cdot\gamma_{M1}\cdot\gamma_n} + \frac{b_b\cdot t_b^2\cdot f_y\left(t_b\right)}{c_{sf}\cdot\gamma_{M1}\cdot\gamma_n} = 294.676\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cored}\cdot\left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\,kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} = \frac{\eta\cdot f_y\left(t_f\right)\cdot h_{web}\cdot t_w}{\sqrt{3}\cdot\gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} = V_{bw.Rd} + V_{bf.Rd} = 2.125\times 10^3\,kN$

Check:

$\frac{V_{Ed}}{\frac{2}{\gamma_n}\cdot V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2\cdot V_{b.Rd}} = 0.511$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 9.658\times 10^{-3}\,m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{wa.eff}\cdot t_d + n_{sw}\cdot s_{wa.eff}\cdot t_{d.eff} + n_{sw}\cdot t_{sw}\cdot t_{d.eff} + A_b = 380.554\,cm^2$

$z_{tp.f.eff.neg} := \frac{s_{wa.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{sw}\cdot s_{wa.eff}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) + n_{sw}\cdot t_{sw}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) + A_b\cdot\left(h_{tk} - \frac{t_b}{2}\right)}{A_{eff.f.tot}} = 372.817\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.026\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff}\cdot\frac{f_y\left(t_d\right)}{\gamma_M} = 9.196\times 10^3\,kN\cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 9.915\times 10^3\,kN\cdot m$

Moment capacity check when $q_{-}M = 69\cdot\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.986$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The reduced stress method

-Changed top plate stiffener thickness to 4mm

Material

$f_y(t) = \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ 335\text{MPa} & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} = 1$

$E = 210\text{GPa}$

$\nu = 0.3$ $G = \frac{E}{2\cdot(1+\nu)} = 80.769\text{GPa}$

Geometry

Bottom plate

$t_b = 16\text{mm}$ $b_b = 1000\text{mm}$ $A_b = b_b t_b = 0.016\text{m}^2$

Top plate

$t_t = 10\text{mm}$ $b_t = 3500\text{mm}$ $A_t = b_t t_t = 0.035\text{m}^2$

Side plates

$t_s = 12\text{mm}$ $b_s = 389.2\text{mm}$ $A_s = b_s t_s = 4.67\cdot 10^{-3}\text{m}^2$

Webs

$t_w = 12\text{mm}$ $b_w = 1340\text{mm}$ $A_w = b_w t_w = 0.016\text{m}^2$

$b_{w.n} = 1250\text{mm}$ $b_{w.p} = 482.8\text{mm}$

Top stiffeners

$t_{st} = 4\text{mm}$ $b_{st} = 140\text{mm}$ $A_{st} = b_{st} t_{st} = 5.6\cdot 10^{-4}\text{m}^2$

$s_{st} = 350\text{mm}$ $n_{st} = 9$

Web stiffeners

$t_{wt} = 12\text{mm}$ $b_{wt} = 120\text{mm}$ $A_{wt} = b_{wt} t_{wt} = 1.44\cdot 10^{-3}\text{m}^2$

$s_{wt} = 440\text{mm}$ $n_{wt} = 3$

Whole bridge

$L_{bridge} = 18\text{m}$ $b_b = \frac{b_t}{2} + b_{w.p} + b_s + t_s = 0.89\text{m}$

$s_{fs} = 2\text{m}$

$I_{sst} = 0.013391\text{m}^4$ From grasshopper

$y_{sg} = 0.485685\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_s = 39\cdot\frac{kN}{m^2}$

$Q = q_s s_{fs} = 136.5\frac{kN}{m}$

$M_{Ed} = \frac{Q\, L_{bridge}^2}{8} = 5.528\cdot 10^3\,\text{kN m}$

$V_{Ed} = \frac{Q\, L_{bridge}}{2} = 1.228\cdot 10^3\,\text{kN}$

Top plate

$\sigma_{x.Ed.tp} = \frac{M_{Ed}}{I_{sst}}\left(h_w - \frac{t_t}{2} - y_{sg}\right) = 164.438\,\text{MPa}$ $\sigma_{z.Ed.tp} = 0$

$S_{sg} = \frac{t_t b_{st}}{n_{st}+2}\left(h_w - \frac{t_t}{2} - y_{sg}\right) = 1.267\cdot 10^{-3}\,\text{m}^3$

$\tau_{Ed.tp} = \frac{V_{Ed} S_{sg}}{I_{sst}\min(t_t; t_{st})} = 29.067\,\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} = \frac{M_{Ed}}{I_{sst}}\left(h_w - t_t - y_{sg}\right) = 162.374\,\text{MPa}$

$\sigma_{z.Ed.sp} = \frac{0}{2 t_s} = 5.688\,\text{MPa}$

$S_{sg} = (h_w - t_t - y_{sg}) t_s \frac{(h_w - t_t - y_{sg})}{2} + \frac{t_t b_t}{n_{st}+2}\left(h_w - \frac{t_t}{2} - y_{sg}\right) + \frac{t_{wt} b_{wt}}{2}\left(h_w - t_t - \frac{b_{wt}}{2} - y_{sg}\right) = 8.713\cdot 10^{-3}\,\text{m}^3$

$S_{tps} = \frac{\left(t_{wt} b_{wt} - 2\frac{t_{wt} b_{wt}}{2}\right)}{n_{wt}+2}\left(h_t - t_t - \frac{b_{wt}}{4} - y_{sg}\right) + \frac{t_t b_t}{n_{st}+2}\left(h_w - \frac{t_t}{2} - y_{sg}\right) = 1.404\cdot 10^{-3}\,\text{m}^3$

Top plate stiffeners

$\tau_{Ed.tps} = \frac{V_{Ed} S_{tps}}{I_{sst} t_{st}} = 32.205\,\text{MPa}$ 162.374 MPa

Governing slenderness from FE-software

$\sigma_{V.1.NEd} = 6.96\cdot\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

$\sigma_{cr} = \sigma_{V.1.NEd}\, q_s = 271.44\,\text{MPa}$

$\alpha_{ult} = \frac{f_y(t_b)}{\sigma_{cr}} = 1.308$

$\alpha_{cr.1.NEd} = 48.766\cdot\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} = \frac{\alpha_{cr.1.NEd}}{q_s} = 1.249$

$\lambda_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 1.023$ Slenderness for all parts

Top plate check

$\psi_{tp} = 1$

$\rho_{p.tp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.767$

Plate-like behaviour [A.1]

$\sigma_{E.tp} = \frac{\pi^2 E t_t^2}{12\cdot(1-\nu^2)\cdot b_{st}^2} = 1.549\,\text{MPa}$

$I_{y.tp} = \frac{b_t t_t^3}{12\cdot(1-\nu^2)} = 3.205\cdot 10^{-7}\,\text{m}^4$

$y_{sg.tp} = \frac{A_{st}\frac{t_t}{2} + n_{st} A_{wt}\left(t_t + \frac{h_{wt}}{2}\right)}{A_{st} + n_{st} A_{wt}} = 14.441\,\text{mm}$

$I_{sg.tp} = \frac{b_t t_t^3}{12} + A_{st}\left(y_{sg.tp} - \frac{t_t}{2}\right)^2 + \ldots = 3.331\cdot 10^{-7}\,\text{m}^4$
$\quad + n_{st}\left[\frac{t_{wt} h_{wt}^3}{12} + A_{wt}\left(t_t + \frac{h_{wt}}{2} - y_{sg.tp}\right)^2\right]$

$\gamma_{tp} = \frac{I_{sst}}{I_{y.tp}} = 103.912$

$\delta_{tp} = \frac{n_{wt} A_{wt}}{A_{st}} = 0.144$

$\alpha_{tp} = \min\left(\frac{s_{fs}}{b_{st}}; 0.5\right) = 0.571$

$k_{\sigma.p.tp} = \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\,(\gamma_{tp}-1)\,(1+\delta_{tp})} & \text{if } \alpha_{tp} \ge \sqrt{\gamma_{tp}} = 280.207 \\ \frac{4\,(1+\sqrt{\gamma_{tp}})}{(\gamma_{tp}-1)\,(1+\delta_{tp})} & \text{if } \alpha_{tp} < \sqrt{\gamma_{tp}} \end{cases}$

$\sigma_{cr.p.tp} = k_{\sigma.p.tp}\,\sigma_{E.tp} = 434.15\,\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1.tef.tp} = \frac{3-\psi_{tp}}{5-\psi_{tp}} t_{tw} = 0.175\text{m}$

$b_{2.tef.tp} = \frac{2}{5-\psi_{tp}} t_{tw} = 0.175\text{m}$ Parts of the top plate that contributes to the stiffener

$b_{st.1.tp} = b_{1.tef.tp} + b_{2.tef.tp} = 0.35\text{m}$

$y_{sg.st.1.tp} = \frac{t_t b_{st.1.tp}\frac{t_t}{2} + A_{wt}\left(t_t + \frac{h_{wt}}{2}\right)}{t_t b_{st.1.tp} + A_{wt}} = 0.015\text{m}$ combined center of gravity for stiffener and contributing top plate.

$I_{st.1.tp} = \frac{b_{st.1.tp} t_t^3}{12} + b_{st.1.tp} t_t\left(y_{sg.st.1.tp} - \frac{t_t}{2}\right)^2 + \frac{t_{wt} h_{wt}^3}{12} + A_{wt}\left(t_t + \frac{h_{wt}}{2} - y_{sg.st.1.tp}\right)^2$ Combined moment of inertia

$\sigma_{cr.st.tp} = \frac{\pi^2 E I_{st.1.tp}}{A_{st.1.tp} s_{fs}} = 467.022\,\text{MPa}$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.p} = \sigma_{cr.st}$

$\sigma_{cr.c.tp} = \sigma_{cr.st.tp}$

Reduction factor for column-like behaviour

$i_{tp} = \sqrt{\frac{I_{st.1.tp}}{A_{st.1.tp}}} = 0.03\text{m}$

$e_{tp} = \max\left(y_{sg.st.1.tp} - t_t - y_{sg} + \frac{h_{wt}}{2} - y_{sg.st.1.tp}\right) = 0.065\text{m}$

$\alpha_{e.tp} = 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.684$ stiffener with open cross section (curve c)

$\phi_{tp} = 0.5\left[1 + \alpha_{e.tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 1.305$ [6.3.1.2]

$\chi_{tp} = \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}; 1\right) = 0.473$

Interpolation

$\psi_{tp} = \min\left(\max\left(\frac{\sigma_{cr.c.tp}}{\sigma_{cr.c.tp}} - 1.0\right); 1\right) = 0$

$\rho_{c.tp} = \begin{cases} \left[\left(\rho_{p.tp} - \chi_{tp}\right) b_{tp}\left(2 - b_{tp}\right) + \chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{p.tp} - \chi_{tp}\right) b_{tp}\left(2 - t_{wt}\right) + \chi_{tp}\right] t_{wt}}{s_{fs}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p.tp} - \chi_{tp}\right) b_{tp}\left(2 - t_{wt}\right) + \chi_{tp}\right] t_{wt}}{s_{fs}} \ge 1 \end{cases} = 0.473$

Shear

$\eta_1 = 1.2$ 7.1 (2)

$\chi_w = \begin{cases} \eta_1 & \text{if } \lambda_p < \frac{0.83}{\eta_1} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_1} < \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p > 1.08 \end{cases} = 0.811$

$\frac{\sigma_{x.sg} s_{wt}}{\gamma_{M1}} = 0.618$

Side plates check

x-direction

$\sigma_{max} = f_y(t_s)$

$\sigma_{CG} = \left[\frac{-\sigma_{max}}{f_y(t_s)} + 1 + \sigma_{max}\right] - 1$

$\psi_{sp.x} = \frac{\sigma(h_w - t_t - y_s)}{\sigma(h_w - t_t - y_s)} = 0.01$

$\sigma_{x.sg} = \frac{M_{Ed}}{I_{sst}}\left(h_w - t_t - y_{sg}\right) = 162.374\,\text{MPa}$

$\sigma_{E.sg.x} = \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2) b_s^2} = 180.432\,\text{MPa}$

$\psi_{spl.x} = \max(3 + \psi_{sp.x}; 0) = 3.01$ $\psi_{sp1} = \frac{\sigma_{x.bot}}{\sigma_{x.sg}} = 0.01$

$\rho_{p.sp.x} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(\psi_{spl.x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.819$

Plate-like behaviour

$k_{\sigma.sp.x} = \begin{cases} 1 & \text{if } \psi_{sp.x} \ge 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29\,\psi_{sp.x} + 9.78\,\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\,(1 - \psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} > -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 7.732$

$\sigma_{cr.sp.x} = \sigma_{E.sp.x} k_{\sigma.sp.x} = 1.395\cdot 10^3\,\text{MPa}$

Column-like behaviour

$\sigma_{cr.c.sp.x} = \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2) b_{ss}^2} = 6.833\,\text{MPa}$

$\alpha_{sp} = 0.21$

$\phi_{sp} = 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 1.11$

$\chi_{sp} = \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}; 1\right) = 0.649$

Interpolation

$\psi_{sp.x} = \min\left(\max\left(\frac{\sigma_{cr.sp.x}}{\sigma_{cr.c.sp.x}} - 1.0\right); 1\right) = 1$

$\rho_{c.sp.x} = \begin{cases} \left[\left(\rho_{p.sp} - \chi_{sp}\right) b_{sp}\left(2 - b_{sp}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right) b_{sp}\left(2 - t_{wt}\right) + \chi_{sp}\right] t_{wt}}{s_{fs}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right) b_{sp}\left(2 - t_{wt}\right) + \chi_{sp}\right] t_{wt}}{s_{fs}} \ge 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} = 1$

$\alpha_{p.sp} = 0.34$ $\lambda_{p0.sp} = 0.7$

$\phi_{sp.sp} = 0.5\left[1 + \alpha_{sp.sp}\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p^2\right] = 1.067$

$\rho_{p.sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{sp.sp} + \sqrt{\phi_{sp.sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.712$

Plate-like behaviour

$k_{\sigma.p.sp.z} = 4$

$\sigma_{E.sp.z} = \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2) b_{ss}^2} = 6.833\,\text{MPa}$

$\sigma_{cr.p.sp.z} = k_{\sigma.p.sp.z}\,\sigma_{E.sp.z} = 27.331\,\text{MPa}$

Column-like behaviour

$\sigma_{cr.c.sp.z} = \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2) b_{ss}^2} = 180.432\,\text{MPa}$

Interpolation

$\psi_{sp.z} = \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1.0\right); 1\right) = 0$

$\rho_{c.sp.z} = \begin{cases} \left[\left(\rho_{p.sp} - \chi_{sp}\right) b_{sp}\left(2 - b_{sp}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right) b_{sp}\left(2 - t_{wt}\right) + \chi_{sp}\right] t_{wt}}{s_{fs}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right) b_{sp}\left(2 - t_{wt}\right) + \chi_{sp}\right] t_{wt}}{s_{fs}} \ge 1 \end{cases} = 0.649$

Top plate stiffeners - check

x-direction

$\rho_{p,c,sps} := \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 0.798$

$\psi_{sps,x} := \frac{\sigma_{1,sps,x} \cdot (z_g - y_{sps})}{\sigma_{1,sps,x} \cdot z_g} = 0.644$

$\sigma_{E,sps,x} := \frac{\pi^2 \cdot E \cdot t_{sps}^2}{12 \cdot (1 - \nu^2) \cdot b_{sps}^2} = 154.939 \cdot MPa$

$k_{\sigma,sps,x} := \begin{cases} 1 & \text{if } \psi_{sps,x} \geq 1 \\ \frac{8.2}{(1.05 + \psi_{sps,x})} & \text{if } 1 > \psi_{sps,x} > 0 \\ 7.81 & \text{if } \psi_{sps,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sps,x} + 9.78 \cdot \psi_{sps,x}^2 & \text{if } 0 > \psi_{sps,x} > -1 \\ 23.9 & \text{if } \psi_{sps,x} = -1 \\ 5.98 \cdot (1 - \psi_{sps,x})^2 & \text{if } -1 > \psi_{sps,x} > -3 \\ 95.68 & \text{if } \psi_{sps,x} < -3 \end{cases} = 4.84$

$\sigma_{cr,sp,sps,x} := k_{\sigma,sps,x} \cdot \sigma_{E,sps,x} = 749.976 \cdot MPa$

Column-like behaviour

$\sigma_{cr,c,sps,x} := \frac{\pi^2 \cdot E \cdot t_{sps}^2}{12 \cdot (1 - \nu^2) \cdot b_{sps}^2} = 0.759 \cdot MPa$

$\chi_{sps} := \chi_{sp}$

Interpolation

$\xi_{sps,x} := \min\left(\max\left(\frac{\sigma_{cr,c,sps,x}}{\sigma_{cr,c,sps,x}} - 1.0\right), 1\right) = 1$

$\rho_{c,sps,x} := \begin{cases} \left[\left(\rho_{sps,sps} - \chi_{sps}\right) \cdot \psi_{sps,x} \cdot \left(2 - \psi_{sps,x}\right) - 1\right] \cdot \xi & \text{if } \frac{\left[\left(\rho_{sps,sps} - \chi_{sps}\right) \cdot \psi_{sps,x} \cdot \left(2 - \psi_{sps,x}\right) - 1\right] \cdot \gamma_{M1}}{\chi_{sps}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{sps,sps} - \chi_{sps}\right) \cdot \psi_{sps,x} \cdot \left(2 - \psi_{sps,x}\right) + \chi_{sps}\right] \cdot \sigma_{sds}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Top plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_{sps} \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.99$

Side plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sps,z} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sps,z} \cdot f_y}{\gamma_{M1}}}\right) + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_{sps} \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.239$

Top plate stiffeners check

$\left(\frac{\sigma_{x,Ed,sps}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sps}}{\frac{\chi_{sps} \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.247$

EWM - Changed top plate stiffener thickness

This chapter covers the EWM when changing the top plate stiffener thickness to 6 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s.so} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350\,mm$

Side plate: $b_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140mm$ $t_{su} := 6mm$ $s_{su} := 6mm$

Stiffeners under: $h_{sl} := 120mm$ $t_{sl} := 12mm$ $s_{sl} := 5mm$

$h_{sw.y} := h_{sw}\cdot\cos\left(21.12\cdot\frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{tx} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{tx} := 2000mm$ $s_{ty0} := 500mm$

$t_{tx.1} := 18mm$ $t_{tx2} := 12mm$ $t_{ty3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d\cdot b_d$ $A_s := 2t_s\cdot h_s$ $A_w := 2t_w\cdot h_w$ $A_b := t_b\cdot b_b$

$A_{su} := n_{su}\cdot t_{su}\cdot h_{su}$ **steners:** $A_{sl} := n_{sl}\cdot t_{sl}\cdot h_{sl}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.109\,m^2$

$A_c := A_{su} + A_d = 0.043\,m^2$ EC1993-1-1 tab. 3.1

$f(t) := \frac{235MPa}{t}$

Changed center of gravity, z_{tp}

$$z_{tp} := \frac{A_d\frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_b\left(t_d + h_s + h_{wy} + \frac{t_b}{2}\right) + A_{sl}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{wy}}{2}\right]}{A_{tot}} = 395.911\,mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\,mm$

Stiffeners overlrudor:

$$I_{y.st.so} := n_{so}\cdot\frac{t_{su}\cdot h_{su}^3}{12} + n_{su}\cdot t_{su}\cdot h_{su}\left(t_d + \frac{h_{su}}{2} - z_{tp}\right)^2 = 7.668\times10^{-4}\,m^4$$

$I_{y.st.su.1} := 6\cdot1.506\cdot10^{-5}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su}\cdot h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2 + 2t_{su}\cdot h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2 + 2t_{su}\cdot h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2 = 7.577\times10^{-4}\,m^4$$

Web:

$I_{y.w.1} := 2\cdot0.000312m^4$ from FEM

$$I_{y.w} := I_{y.w.1} + 2h_w\left[h_s - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 2.55\times10^{-3}\,m^4$$

Deck:

$$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d\left(\frac{t_d}{2} - z_{tp}\right)^2 = 5.349\times10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b} := \frac{b_b t_b^3}{12} + b_b t_b\left(h_{tx} - z_{tp} - \frac{t_b}{2}\right)^2 = 3.781\times10^{-3}\,m^4$$

Side plate:

$$I_{y.s} := 2\cdot\frac{t_s h_s^3}{12} + 2t_s h_s\left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 4.598\times10^{-4}\,m^4$$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.014\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}}\cdot z - \sigma_{max}$

$\sigma_{1.neg}(\tau_1,\tau_2) := \max(\sigma_{neg}(\tau_1),\sigma_{neg}(\tau_2))$

$\sigma_{2.neg}(\tau_1,\tau_2) := \min(\sigma_{neg}(\tau_1),\sigma_{neg}(\tau_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}}\cdot z + \sigma_{max}$

$\sigma_{1.pos}(\tau_1,\tau_2) := \max(\sigma_{pos}(\tau_1),\sigma_{pos}(\tau_2))$

$\sigma_{2.pos}(\tau_1,\tau_2) := \min(\sigma_{pos}(\tau_1),\sigma_{pos}(\tau_2))$

Buckling factor

$\psi_{neg}(\tau_1,\tau_2) := \frac{\sigma_{2.neg}(\tau_1,\tau_2)}{\sigma_{1.neg}(\tau_1,\tau_2)}$ $\psi_{pos}(\tau_1,\tau_2) := \frac{\sigma_{2.pos}(\tau_1,\tau_2)}{\sigma_{1.pos}(\tau_1,\tau_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1-\psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(b_w,b,t) := \frac{\frac{b_w}{t}}{28.4\cdot\varepsilon(t)\cdot\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p,\psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left(\frac{\lambda_p - 0.055\max((3+\psi),0)}{\lambda_p^2},1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2) Internal parts

$$\rho_{4.3}(\lambda_p,\psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2},1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -8.522\times10^{-3}$

$$\text{Condition} := \begin{cases} 62.1\left(t_s\right)\left(1-\psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42.\varepsilon(t_s)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$$

$$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 7.864$

$s_{c.so} := s_{so} - s_{su}$ Table 5.2 ES1993-1-1

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s),h_s,t_s) = 0.501$

$\rho_s := \rho_{4.2}(\lambda_{p.s},\psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 3.289\times10^{-3}\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 385.911\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 385.911\,mm$

$h_{s.eff.c1} := 0.4\,h_{s.eff.c} = 154.364\,mm$

$h_{s.eff.c2} := 0.6\,h_{s.eff.c} = 231.547\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\,mm$

$A_{s.eff.c} := 2\cdot h_{s.eff.c}\cdot t_s = 9.262\times10^{-3}\,mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t})\cdot t_s = 9.341\times10^{-3}\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s)\frac{E\cdot\pi^2\cdot t_s^2}{12\left(1-\nu^2\right)\cdot b_s^2} = 1.419\times10^3\,MPa$ EC1993-1-5 eq. A.1

Stiffeners

$t_{sl} := t_d = 10\,mm$

$h_{sw} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{sup}(z_2)}{\sigma_{sup}(z_1)} = 0.637$

$k_{\sigma.4.2}(\psi_{so}) = 0.591$ Higher stress close to support

Class$_{so}$:= "Class4" if $\frac{h_{so}}{t_{so}} > 21 \cdot \epsilon(\psi_{so}) \cdot \sqrt{k_{\sigma.4.2}(\psi_{so})}$ = "Class4"
"Lower than class 4" otherwise

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2}(\psi_{so}), b_{so}, t_{so}) = 1.313$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_{so}) = 0.653$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 91.358\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 4.933 \times 10^{-3}\ m^2$

Deck

$\frac{h_{so}}{t_d} = 35 \quad 42 \cdot \sqrt{\frac{235MPa}{f_d(t_d)}} = 34.172$

Class$_d$:= "Class4" if $\frac{h_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_d(t_d)}}$ = "Class4"
"Lower than class 4" otherwise

$z_d := -\frac{t_d}{2} = -5 \times 10^{-3}\ m$ $\psi_d := \psi_{sup}(z_d, z_d) = 1$

$k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$ [tab. 5.2 ES1993-1-1] $k_{sw} := k_{so} - 1$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, t_{cm}, t_d) = 0.744$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.946$ Internal cross-sectional parts

$h_{so.eff} := \rho_d \cdot t_{cm} = 325.55\ mm$

Areas:

$A_{sw} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 3.697 \times 10^{-3}\ m^2$

$A_{c.eff.loc} := n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 3.477 \times 10^{-3}\ m^2$

More than three longitudinal stiffeners -> k_{sw} obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-2}\ m^4$

$y_{sp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + b_d \cdot t_d} = 18.322\ mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl}\right)^2\right] \dots = 4.761 \times 10^{-7}\ m^4$
$+ \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl}\right)^2\right]$

$\gamma := \frac{I_{sl}}{I_p} = 148.546$ $\alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 7.56 \times 10^{-3}\ m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035\ m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.216$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[(1 + \alpha^2)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt{1} \\ \dots & \text{if } \alpha > \sqrt{1} \end{cases} = 376.027$

$\lambda_{p.p} := \sqrt{\frac{\beta_{A.c} \cdot f_d(t_d)}{\sigma_{cr.p}}} = 0.757$ herwise

$\rho := \rho_{4.2}(\lambda_p, \psi) = 0.937$ Plate-like behaviour

Effective area of the orthotropic plate: EC1993-1-5 eq.A.1

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + n_{so} \cdot t_{so} \cdot t_d = 3.584 \times 10^{-3}\ m^2$

Buckling reduction factor $\chi_c := \frac{A_{c.eff.loc}}{A_{c.eff}} = 0.94$

$N_{Rd} := A_{c.eff} \cdot \frac{f_d(t_d)}{\gamma_M} = 1.272 \times 10^4\ kN$

Column like behaviour

$y_{sp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + n_{so} \cdot t_d} = 19.516\ mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl.c}\right)^2\right] \dots = 5.212 \times 10^{-6}\ m^4$
$+ \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl.c}\right)^2\right]$

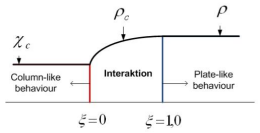
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + n_{so} \cdot t_d = 4.34 \times 10^{-3}\ m^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + n_{so} \cdot t_d = 3.804 \times 10^{-3}\ mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.876$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

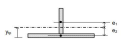
$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 622.221\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_d(t_{so})}{\sigma_{cr.sl}}} = 0.707$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections



$e = 2.718$

$i := \sqrt{\frac{I_{sl.c}}{A_{sl}}} = 104.739\ mm$

$\bar{\lambda} := \max\left[\left(y_{sp.sl} - \frac{t_d}{2}\right) \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl}\right)\right] = 61.678\ mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{\bar{\lambda}}} = 0.543$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.888$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.702$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cp.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.702$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot t_{so} \cdot t_{so.eff} \cdot t_d \cdot \sigma_{sup}\left(t_d \cdot \frac{t_d}{2}\right) + A_{so} \cdot \sigma_{sup}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.258 \times 10^4\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot t_{so.eff} \cdot t_d + A_{so}} = -341.196\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_d(t_{so})}{\gamma_M} = 249.239\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_d(t_{so})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{so} \cdot t_d \cdot t_{so.eff} = 0.033\ m^2$

$A_{tot.eff} := A_{sw.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{s.eff} = 0.104\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} & A_{d.eff} \cdot \frac{t_d}{2} + A_{sw.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ & + 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ & \left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \dots \\ & + A_w \cdot \left(t_d + h_s + \frac{h_{sw.y}}{2}\right) + A_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{so0}) - \frac{h_{sw.y}}{2}\right] \dots \\ & + A_b \cdot \left(h_{tb} - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 411.984\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right.$
$+ \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2\right] \dots$
$\left. + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right]\right] = 5.196 \times 10^{-4}\ m^4$

$I_{y.d.eff} := \frac{n_{so} \cdot t_{so.eff} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot t_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.393 \times 10^{-7}\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.d.eff} = 0.014\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{td.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 3.477 \times 10^{-3}\ m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_d(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -3.17 \times 10^{-3}\ m^2$

$A_{sd.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{sw.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + n_{so} \cdot t_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + n_{so} \cdot t_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 276.688\ cm^2$

$A_{c.eff.loc.mod}$ Lower stiffener in tension -> no reduction

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with reduction factors.

$t_{d.eff} := \begin{cases} \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.021\ mm$

$y_{sw.eff} := \frac{\begin{aligned} & n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) \dots \\ & + n_{so} \cdot t_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ & + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ & + A_w \cdot \left(t_d + h_s + \frac{h_{sw.y}}{2}\right) + A_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{so0}) - \frac{h_{sw.y}}{2}\right] \dots \\ & + A_b \cdot \left(h_{tb} - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 444.482\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so} \cdot t_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{so} \cdot t_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 6.288 \times 10^{-3}\ m^4$

$z_{sw.fin} := \sin\left(21.12 \cdot \frac{\pi m}{180}\right) \cdot s_{so0} = 7.206\ mm$

Stiffeners overlunder:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 7.492 \times 10^{-4}\ m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots = 5.562 \times 10^{-4}\ m^4$
$+ 2 \cdot n_{so} \cdot h_{so} \cdot \left[t_d + h_s + \frac{1}{2} \cdot (h_{wy} - s_{so0y}) - \frac{h_{sw.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$
$+ 2 \cdot n_{so} \cdot h_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{so0y}) - \frac{h_{sw.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$
$+ 2 \cdot n_{so} \cdot h_{so} \cdot \left[t_d + h_s + (h_{wy} - s_{so0y}) - \frac{h_{sw.y}}{2} - z_{tp.eff.neg}\right]^2$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot n_w \cdot h_w \cdot \left[h_{tb} - \frac{h_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp.eff.neg}\right]^2 = 1.861 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{h_b \cdot b_b^3}{12} + t_b \cdot h_b \cdot \left(h_{tb} - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 3.063 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right.$
$+ \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2\right] \dots$
$\left. + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right]\right] = 6.554 \times 10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.317 \times 10^{-2}\ mm^4$

Load

$q_M := 74 \frac{kN}{m^2}$

$Q_M := q_M \cdot b_d = 259\ \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.049 \times 10^4\ kN \cdot m$

$V_{Ed} := \frac{Q_M \cdot l_{bridge}}{2} = 2.331 \times 10^3\ kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.03\ m^3$

$M_{Rd} := W_{y.eff} \frac{f_y(t_f)}{\gamma_{M1}} = 1.052 \times 10^4\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.997$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_{sw}) \cdot t_{sw} = 0.293 m$

$\alpha_w := \min\left(\frac{b_{sw}}{b_w}, \frac{180}{\pi}\right) = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot b_{sw}}{2} = 0.503 m$

$a_{st.z.2} := a_{st.z.1} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913 m$

$a_{st.z.3} := a_{st.z.2} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324 m$

Moment of inirtia stiffeners z-direction (two stiffeners)

$I_{st.z} := 2 \cdot t_{sw} \cdot b_{sw} \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\ m^4$

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$a_{w.z.1} := \frac{b_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} = 0.525 m$

$a_{w.z.1} := a_{st.z.1} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935 m$

$a_{w.z.2} := a_{st.z.2} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346 m$

Moment of inirtia of 15st of the web z-direction

$I_{sw.z} := 2 b_{web} \cdot t_{sw} \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$

$I_{st.z.tot} := I_{st.z} + I_{sw.z} = 0.029 m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34 \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.491$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{sti} = 0$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4 \cdot t_w \cdot \varepsilon(t_w) \sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[\left[\left(\frac{h_w}{a}\right)^2 \sqrt[4]{\frac{I_{st.z.tot}}{\left(\min\left(t_f, t_w\right)\right)^3 h_w}}\right], \frac{2.1}{\min\left(t_f, t_w\right)} \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4 \left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \frac{I_{st.z.tot}}{\min\left(t_f, t_w\right)^3 h_{web}} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_f, t_w\right)^3 h_{web}}} & \text{otherwise} \\ 4.1 + \frac{}{\alpha^2} \end{cases} = 1.635 \times 10^3$

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w) \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6) ,has to be performed for stiffened webs:

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ SS-EN 1993-1-5 5.3 Table 5.1, stiff end stiffener.

$\frac{t}{t_w} \leq 31 \frac{\varepsilon(t_f)}{\eta} \sqrt{k_{\tau 1}} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

Kapacity of the web

$\gamma_{M1} = 1.00$ $\gamma_{M0} = 1.2$ Safety class 3

$\gamma_{M1} = 1.00$

SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_f) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3\ kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.331 \times 10^3\ kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.548$ (contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_f) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{pf.eff.neg} := \frac{t_d}{2} = 439.482 \cdot mm$

$a_{uf.eff.neg} := h_n - \frac{t_u}{2} - z_{pp.eff.neg} = 437.518 \cdot mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{pf.eff.neg} \left| a_{pf.eff.neg} \right| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \left| a_{uf.eff.neg} \right| \cdot f_y(t_h)}{\gamma_{M0}} = 7.564 \times 10^3\ kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{pf} := a \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.623 m$

$c_{uf} := a \left[0.25 + \frac{1.6 \cdot b_h \cdot t_h^2 \cdot f_y(t_h)}{t_s \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.59 m$

$V_{bf.Rd.cored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{pf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_h \cdot t_h^2 \cdot f_y(t_h)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 294.676 kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cored} \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_f) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{Rd.tot} = V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3\ kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.548$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.tot} := I_{y.d.eff.tot} + I_{y.h.tot} = 9.351 \times 10^{-3}\ m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{wn.eff} \cdot t_d + n_{sw} \cdot s_{wn.eff} \cdot t_{d.eff} + n_{sw} \cdot t_{sw} \cdot t_{d.eff} + A_h = 402.053 \cdot cm^2$

$z_{pp.f.eff.neg} := \frac{s_{wn.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{sw} \cdot s_{wn.eff} \cdot t_{d.eff} \left(\frac{t_{d.eff}}{2}\right) + \dots + n_{sw} \cdot t_{sw} \cdot t_{d.eff} \left(\frac{t_{d.eff}}{2}\right) + A_h \left(h_n - \frac{t_h}{2}\right)}{A_{eff.f.tot}} = 353.233 \cdot mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.tot}}{z_{pp.f.eff.neg}} = 0.026 \cdot m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \frac{f_y(t_f)}{\gamma_M} = 9.398 \times 10^3 \cdot kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.052 \times 10^4 \cdot kN \cdot m$

Moment capacity check when $q_{-}M = 74 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.997$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The reduced stress method

-Changed top plate stiffener thickness to 6 mm

Material

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ (335MPa) & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} := 1$

$E := 210GPa$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot GPa$

Geometry

Bottom plate

$t_b := 16mm$ $b_b := 1000mm$ $A_b := b_b t_b = 0.016m^2$

Top plate

$t_d := 10mm$ $b_d := 3500mm$ $A_d := b_d t_d = 0.035m^2$

Side plates

$t_s := 12mm$ $h_s := 389.2mm$ $A_s := h_s t_s = 4.67\times10^{-3}m^2$

Webs

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w t_w = 0.016m^2$

$h_{w.x} := 1250mm$ $h_{w.y} := 482.8mm$

Top stiffeners

$t_{st} := 6mm$ $h_{st} := 140mm$ $A_{st} := h_{st} t_{st} = 8.4\times10^{-4}m^2$

$s_{st} := 350mm$ $n_{st} := 9$

Web stiffeners

$t_{wt} := 12mm$ $h_{wt} := 120mm$ $A_{wt} := h_{wt} t_{wt} = 1.44\times10^{-3}m^2$

$s_{wt} := 440mm$ $n_{wt} := 3$

Whole bridge

$L_{bridge} := 18m$ $h_h := \frac{h_b}{2} + h_{w.y} + h_s + t_d = 0.89m$ $s_h := 2m$

$I_{tot} := 0.0136\cdot3m^4$ From grasshopper

$Y_{tp} := 0.494174m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 56\frac{kN}{m^2}$

$Q := q\cdot b_d = 196\frac{kN}{m}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 7.938\times10^3 kN\cdot m$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 1.764\times10^3 kN$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\left(h_h - \frac{t_d}{2} - Y_{tp}\right) = 227.23MPa$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{st}+2}\left(h_h - \frac{t_d}{2} - Y_{tp}\right) = 1.244\times10^{-3}m^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_w;t_{wt})} = 26.778MPa$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\left(h_h - t_d - Y_{tp}\right) = 224.323MPa$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 8.167MPa$

$S_{sp} := \left(h_h - t_d - Y_{tp}\right)t_s\left(\frac{h_h - t_d - Y_{tp}}{2} + \frac{t_d\cdot b_d}{n_{st}\cdot\left(t_s + h_{st}\right)}\left(h_h - \frac{t_d}{2} - Y_{tp}\right)\right) = 8.926\times10^{-3}m^3$

$\sigma_{x.Ed.sps} := \frac{M_{Ed}}{I_{tot}}\left(h_h - t_d - Y_{tp}\right) = 224.323MPa$

$S_{sps} := \frac{t_{wt}\cdot h_{wt}}{n_{wt}+2}\left(h_h - t_d - \frac{h_{wt}}{4} - Y_{tp}\right) + 2\frac{t_s\cdot h_{wt}}{n_{wt}+2}\left(h_h - t_d - \frac{h_{wt}}{2} - Y_{tp}\right) + \frac{t_d\cdot b_d}{n_{st}+2}\left(h_h - \frac{t_d}{2} - Y_{tp}\right) = 1.418\times10^{-3}m^3$

$\tau_{Ed.sps} := \frac{V_{Ed}\cdot S_{sps}}{I_{tot}\cdot t_{wt}} = 30.528MPa$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 5.47\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

79.1

$\sigma_V := \sigma_{V.UNIT}\cdot q = 306.32MPa$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.159$

$\sigma_{cr.UNIT} := 119.87\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\sigma_{cr.UNIT}}{q} = 2.141$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.736$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.953$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot\left(1-\nu^2\right)\cdot b_d^2} = 1.549MPa$

$I_{tp} := \frac{b_d\cdot t_d^3}{12\cdot\left(1-\nu^2\right)} = 3.205\times10^{-7}m^4$

$y_{tp.sl.tp} := \frac{A_d\frac{t_d}{2} + n_{wt}\cdot A_{wt}\left(t_d + \frac{h_{wt}}{2}\right)}{A_d + n_{wt}\cdot A_{wt}} = 18.322mm$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + \ldots + n_{wt}\left[\frac{t_{wt}\cdot h_{wt}^3}{12} + A_{wt}\left(t_d + \frac{h_{wt}}{2} - y_{tp.sl.tp}\right)^2\right] = 4.761\times10^{7}mm^4$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 148.546$

$\delta_{tp} := \frac{n_{wt}\cdot A_{wt}}{A_d} = 0.216$

$\alpha_{tp} := \max\left(\frac{s_{st}}{b_d};0.5\right) = 0.571$ $\sqrt{\gamma_{tp}} = 3.491$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\alpha_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt{\gamma_{tp}} \\ \frac{4\left(1+\sqrt{\gamma_{tp}}\right)}{(\alpha_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt{\gamma_{tp}} \end{cases} = 376.027$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 582.611MPa$

Column-like behaviour [4.5.3]

$b_{1.ind.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}s_{tn} = 0.175m$

$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}s_{tn} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.ind.tp} + b_{2.sup.tp} = 0.35m$

$y_{tp.sl.1.tp} := \frac{t_d\frac{t_d}{2} + A_{wt}\left(t_d + \frac{h_{wt}}{2}\right)}{t_d\cdot b_{sl.1.tp} + A_{wt}} = 0.02m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^3}{12} + b_{sl.1.tp}t_d\left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{wt}\cdot h_{wt}^3}{12} + A_{wt}\left(t_d + \frac{h_{wt}}{2} - y_{tp.sl.1.tp}\right)^2$

Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{tn}^2} = 622.221MPa$ critical stress for kolummlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.035m$

$e_{tp} := \max\left(Y_{tp.sl.1.tp} - t_d - t_d + \frac{h_{wt}}{2} - y_{tp.sl.1.tp}\right) = 0.06m$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.647$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e.tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.944$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\left(\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}\right)};1\right) = 0.651$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1,0\right);1\right) = 0$

$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.651$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.128$

$\frac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.755$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}z + \sigma_{max}\right]\cdot1$

$\psi_{sp.x} := \frac{\sigma(h_h - t_d - h_s)}{\sigma(h_h - t_d)} = -8.745\times10^{-3}$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{sp.x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1.055$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\left(h_h - t_d - Y_{tp}\right) = 224.323MPa$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -8.745\times10^{-3}$

Plate-like behaviour

$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 180.432MPa$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} \equiv 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} \equiv 0 \\ 7.81 - 6.29\cdot\psi_{sp.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} \equiv -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 7.866$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.419\times10^3 MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 6.833MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.827$

$\chi_{sp} := \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)};1\right) = 0.83$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1,0\right);1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

z-direction

$\psi_{sp.z} := 1$

$\sigma_{p.sp} := 0.34$ $\lambda_{p0.sp} := 0.7$

$\alpha_{p.sp} := 0.5\left[1 + \alpha_{p.sp}\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.874$

$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.96$

Plate-like behaviour

$k_{\sigma.p.sp.z} = 4$

$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 6.833MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 27.331MPa$

Column-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 180.432MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1,0\right);1\right) = 0$

$$\rho_{c,sp,x} := \begin{cases} 1 & \text{if } \left[\frac{\left[\left(\rho_{pc,sp} - \chi_{sp}\right)\left(\varepsilon_{sp,x}\left(2 - \xi_{sp,x}\right) - \right]\right]\sigma_{alt}}{\gamma_{M1}} < 1 \right. \\ \left. \frac{\left[\left(\rho_{pc,sp} - \chi_{sp}\right)\left(\varepsilon_{sp,x}\left(2 - \xi_{sp,x}\right) + \chi_{sp}\right]\sigma_{alt}}{\gamma_{M1}} \geq 1 \right. \end{cases} = 0.83$$

Top plate stiffeners - check

x-direction

$$\rho_{px,tps} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{tps,x} := \frac{\sigma_2\left(h_n - t_{fl} - h_{wb}\right)}{\sigma_2\left(h_n - t_{fl}\right)} = 0.637$$

$$\sigma_{E,tps,x} := \frac{\pi^2 \cdot E \cdot t_{tps}^2}{12\left(1 - \nu^2\right)b_{tps}^2} = 348.612 \cdot MPa$$

$$k_{\sigma,tps,x} := \begin{cases} 1 & \text{if } \psi_{tps,x} \equiv 1 \\ \frac{8.2}{\left(1.05 + \psi_{tps,x}\right)} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{tps,x} \equiv 0 \\ 7.81 - 6.29\psi_{tps,x} + 9.78\psi_{sp,x}^2 & \text{if } 0 > \psi_{tps,x} > -1 \\ 23.9 & \text{if } \psi_{tps,x} \equiv -1 \\ 5.98\left(1 - \psi_{tps,x}\right)^2 & \text{if } -1 > \psi_{sp,x} \geq -3 \\ 95.68 & \text{if } \psi_{tps,x} < -3 \end{cases} = 3.803$$

$$\sigma_{cr,p,tps,x} := k_{\sigma,tps,x}\sigma_{E,tps,x} = 1.326 \times 10^3 \cdot MPa$$

Column-like behaviour

$$\sigma_{cr,c,tps,x} := \frac{\pi^2 \cdot E \cdot t_{tps}^2}{12\left(1 - \nu^2\right)b_{tps}^2} = 1.708 \cdot MPa$$

$$\chi_{tps} := \chi_{sp}$$

Interpolation

$$\xi_{tps,x} := \min\left(\max\left(\frac{\sigma_{cr,p,tps,x}}{\sigma_{cr,c,tps,x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c,tps,x} := \begin{cases} 1 & \text{if } \left[\frac{\left[\left(\rho_{px,tps} - \chi_{tps}\right)\left(\varepsilon_{tps,x}\left(2 - \xi_{tps,x}\right) - \right]\right]\sigma_{alt}}{\gamma_{M1}} < 1 \right. \\ \left. \frac{\left[\left(\rho_{px,tps} - \chi_{tps}\right)\left(\varepsilon_{tps,x}\left(2 - \xi_{tps,x}\right) + \chi_{tps}\right]\sigma_{alt}}{\gamma_{M1}} \geq 1 \right. \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,tps}}{\frac{\rho_{c,tps} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,tps}}{\frac{\chi_{tw} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right)^2 = 0.979$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sp,z} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sp,z} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right) ... = 0.426$$
$$+ 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_{tw} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,tps}}{\frac{\rho_{c,tps,x} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,tps}}{\frac{\chi_{tw} \cdot t_{fl}\left(t_{fl}\right)}{\gamma_{M1}}}\right)^2 = 0.417$$

EWM - Changed top plate stiffener thickness

This chapter covers the EWM when changing the top plate stiffener thickness to 8 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s,so} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d,0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s,so}} = 350\,mm$

Side plate: $b_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140mm$ $t_{su} := 8mm$ $s_{su} := 6mm$

Stiffeners under: $h_{sl} := 120mm$ $t_{sl} := 12mm$ $s_{sl} := 5mm$

$h_{sl,y} := h_{sl} \cdot \cos\left(21.12 \frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_h := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_h := 2000mm$ $s_{h0} := 500mm$

$t_{h,1} := 18mm$ $t_{h2} := 12mm$ $t_{h3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d \cdot b_d$ $A_s := 2 t_s \cdot h_s$ $A_w := 2 t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{su} := n_{su} \cdot s_{su} \cdot h_{su} \cdot t_{su}$ **eters:**

$E := 210GPa$

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.111\,m^2$ -1 tab. 3.1

$A_c := A_{sl} + A_d = 0.045\,m^2$ ief

$\varepsilon(\eta) := \sqrt{\frac{235MPa}{f_y(\eta)}}$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d\frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_b\left(t_d + h_s + h_{wy} + \frac{t_b}{2}\right) + A_{sl}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{sl,y}}{2}\right]}{A_{tot}} = 388.753\,mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \frac{\pi}{180}\right) s_{su0} = 7.206\,mm$

Stiffeners over/under:

$I_{y,st.su} := n_{su} \frac{t_{su} h_{su}^3}{12} + n_{su} t_{su} h_{su}\left(t_d + \frac{h_{su}}{2} - z_{tp}\right)^2 = 9.774 \times 10^{-4}\,m^4$

$I_{y,st.su.1} := 6 \cdot 1.506\,10^{-6}\,m^4$ from FEM

$$I_{y,st.su} := I_{y,st.su.1} + 2 t_{su} h_{su}\left[t_d + h_s + \frac{1}{2}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 + 2 t_{su} h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 + 2 t_{su} h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 = 7.909 \times 10^{-4}\,m^4$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 h_w t_w\left[h_h - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 2.664 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d\left(\frac{t_d}{2} - z_{tp}\right)^2 = 5.155 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b t_b^3}{12} + b_b b_b\left(h_h - z_{tp} - \frac{t_b}{2}\right)^2 = 3.893 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\left(\frac{t_s h_s^3}{12} + 2 t_s h_s\left(t_d + \frac{h_s}{2} - z_{tp}\right)^2\right) = 4.347 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.su} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.014\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi \equiv 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi \equiv 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi \equiv -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ 0.57 & \text{if } \psi \equiv 0 \\ 0.85 & \text{if } \psi \equiv -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$$k_{\sigma.4.2ii}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi \equiv 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi \equiv -1 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(b_w, b, t) := \frac{\frac{b_w}{t}}{28.4 \cdot \varepsilon(\eta) \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max((3 + \psi), 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2) Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.028$

$$\text{Condition} := \begin{cases} 62\,\varepsilon(f_y)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42\,\varepsilon(f_y)}{0.67 + 0.33\,\psi_s} & \text{otherwise} \end{cases}$$

$$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 7.991$

$s_{s.so} := s_{so} - t_{so}$ Table 5.2 ES1993-1-1

$\lambda_{p.s} := \lambda_p(b_{\sigma.4.1}(\psi_s), b_s, t_s) = 0.497$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.01\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 378.753\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s\left(h_{s.c}\right) = 378.753\,mm$

$h_{s.eff.c1} := 0.4 h_{s.eff.c} = 151.501\,mm$

$h_{s.eff.c2} := 0.6 h_{s.eff.c} = 227.252\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\,mm$

$A_{s.eff.c} := 2 h_{s.eff.c} t_s = 9.09 \times 10^3\,mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right) t_s = 9.341 \times 10^3\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \frac{E \cdot \pi^2 \cdot s_s^2}{12\left(1 - \nu^2\right) h_s^2} = 1.442 \times 10^3\,MPa$ EC1993-1-5 eq. A.1

Stiffeners

$b_{st} := t_d = 10\,mm$

$b_{sw} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.63$

$k_{\sigma.4.2}(\psi_{so}) = 0.596$ Higher stress close to support

Class4_so := "Class4" if $\frac{b_{so}}{t_{so}} > 21 \cdot \varepsilon \sqrt{k_{\sigma.4.2}(\psi_{so})}$ = "Class4"
"Lower than class 4" otherwise

$\lambda_{p.so} := \lambda_p(b_{\sigma.4.2}(\psi_{so}), b_{so}, t_{so}) = 0.981$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_s) = 0.824$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so} \cdot b_{so} = 115.335\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot b_{so.eff} = 8.304 \times 10^{3}\ mm^2$

Deck

$\frac{b_{sd}}{t_d} = 35$ $42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ := "Class4" if $\frac{b_{sd}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4"
"Lower than class 4" otherwise

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\ m$ $\psi_d := \psi_{neg}(z_d, z_d) = 1$

$b_{sw} := b_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, b_{sw.cal}, t_d) = 0.74$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.95$ Internal cross-sectional parts

$b_{sd.eff} := \rho_d \cdot b_{sw} = 324.751\ mm$

Areas:

$A_{sw} := n_{so} \cdot t_{so} \cdot t_d + n_{sw} \cdot t_{so} \cdot (b_{so.eff} + t_d) = 4.052 \times 10^{4}\ mm^2$

$A_{c.eff.loc} := n_{so} \cdot t_{so} \cdot t_d + n_{sw} \cdot t_{so} \cdot (b_{so.eff} + t_d) = 3.825 \times 10^{4}\ mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{5}\ mm^4$

$y_{p.sl} := \frac{n_{so} \cdot t_{so} \cdot b_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot b_{so} + b_d \cdot t_d} = 21.77\ mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{p.sl}\right)^2\right] + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{p.sl}\right)^2\right] = 6.078 \times 10^{7}\ mm^4$

$\gamma := \frac{l_{sl}}{I_p} = 189.626$ $\alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot b_{so} \cdot t_{so} = 0.01\ m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035\ m^2$

$\delta_{sl} := \frac{A_{sl}}{A_p} = 0.288$

$k_{\sigma.p} := \begin{cases} \left(2\left[(1+\alpha^2)^2 + \gamma - 1\right]\right) & \text{if } \alpha \le \sqrt{\gamma} \\ \left(\frac{(3+\psi)\cdot\gamma}{\alpha^2} + (1+\psi)\right) & \text{otherwise} \end{cases} = 452.682$

$\lambda_{p.p} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.691$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 0.595$ Plate like behaviour
simple plate:

Effective area of the stiffened plate(deck):

$\sigma_{cr.p} = 701.581\ MPa$

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d = 4.097 \times 10^{3}\ mm^2$ 3-1-5 eq A.1

Buckling resistance $\chi_c := \frac{A_{c.eff.loc}}{A_c} = 0.944$

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.455 \times 10^{4}\ kN$

Column like behaviour

$y_{sp.sl.c} := \frac{n_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot h_{so} + n_{so} \cdot t_d} = 23.182\ mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl.c}\right)^2\right] + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl.c}\right)^2\right] = 6.631 \times 10^{5}\ mm^4$

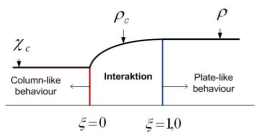
Buckling resistance stiffener

$A_{sw} := t_{so} \cdot h_{so} + n_{so} \cdot t_d = 4.62 \times 10^{3}\ mm^2$

$A_{sl.eff} := t_{so} \cdot b_{so.eff} + n_{so.eff} \cdot t_d = 4.17 \times 10^{3}\ mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.903$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 743.723\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sl})}{\sigma_{cr.sl}}} = 0.656$

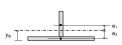
Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 114.696\ mm$

$a_c := \min\left(\left|\left(y_{p.sl} - \frac{t_d}{2}\right)\right|, \left|\left(t_d + \frac{h_{so}}{2} - y_{p.sl}\right)\right|\right) = 58.23\ mm$



$\alpha_c := \alpha + \frac{0.09}{\frac{i}{e}} = 0.536$

$\phi := 0.5\left[1 + \alpha_c \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.838$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.736$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (columnplate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.736$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot t_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(t_d - \frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.32 \times 10^{3}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot t_{so.eff} \cdot t_d + A_{so}} = -335.759\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} = 261.396\ MPa$

$K_{A.2.1}$ = "OK" if $\frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} > \sigma_{com.Ed}$ = "OK"
"EJ OK" otherwise

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.032\ m^2$

$A_{tot.eff} := A_{sw.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{s.eff} = 0.107\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} & A_{d.eff}\left(\frac{t_d}{2} + A_{so.eff}\left(t_d + \frac{h_{so.eff}}{2}\right)\right) \dots \\ & + 2\left[t_s \cdot h_{s.eff.c1}\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ & \left. + t_s \cdot h_{s.eff.c2}\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t}\left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \dots \\ & + A_w\left(t_d + h_s + \frac{h_{wt}}{2}\right) + A_{so}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{so0}\right) - \frac{h_{so.y}}{2}\right] \dots \\ & + A_b\left(h_w - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 401.986\ mm$

$I_{y.s.eff} := 2\left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1}\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2}\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t}\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 4.818 \times 10^{-4}\ m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff}\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.118 \times 10^{-3}\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.d.eff} = 0.014\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{tot.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 3.825 \times 10^{4}\ mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed}\ \gamma_M} = -3.597 \times 10^{3}\ mm^2$

$A_{sl.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{eff.tot} := A_{c.eff} + A_{sw} + A_{so} + A_b = 975.541\ cm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot A_{c.eff.loc.mod}}{\sigma_{com.Ed}\ \gamma_M} \cdot t_{sl}$ if $K_{A.2.1}$ = "EJ OK" $= 314.133\ cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced even further if the web is in tension. Therefore the web-thickness is changed with earlier area reduction factors.

A_{sw} Lower stiffener in tension --> no reduction

$t_{t.eff} := \begin{cases} \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d \end{cases} = 7.365\ mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} & n_{so} \cdot s_{so.eff} \cdot t_{so} \cdot \frac{t_d.eff}{2}\left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ & + n_{so} \cdot s_{so.eff} \cdot t_d.eff\left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1}\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ & + t_s \cdot h_{s.eff.c2}\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t}\left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ & + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{so0}\right) - \frac{h_{so.y}}{2}\right] \dots \\ & + A_b\left(h_w - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 429.644\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff}\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.856 \times 10^{-3}\ m^4$

$s_{so.eff} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{so0} = 7.206\ mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot s_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot s_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.097 \times 10^{-3}\ m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots + 2 \cdot t_{su} \cdot h_{su}\left[t_d + h_s + \frac{1}{2}\left(h_{wy} - s_{so0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot t_{su} \cdot h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{so0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot t_{su} \cdot h_{su}\left[t_d + h_s + \left(h_{wy} - s_{so0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 = 6.134 \times 10^{-4}\ m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\ m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w\left[h_b - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.055 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot h_b^3}{12} + t_b \cdot b_b\left(h_b - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 3.274 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2\left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1}\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2}\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t}\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 5.91 \times 10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.349 \times 10^{-10}\ mm^4$

Load

$q_M := 78\frac{kN}{m^2}$

$Q_M := q_M \cdot b_{tl} = 273\frac{kN}{m}$ $V_{Ed} := \frac{Q_M \cdot b_{bridge}}{2} = 2.457 \times 10^{3}\ kN$

$M_{Ed} := \frac{Q_M \cdot b_{bridge}^2}{8} = 1.106 \times 10^{4}\ kN \cdot m$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.031\ m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.114 \times 10^{4}\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.992$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeners

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot t_{sl} \cdot t_w = 0.293\,m$

$\alpha_w := asin\left(\frac{h_{ww}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ww}}{2} + \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{ss}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$

Moment of inirtia stiffeners z-direction (two stiffeners)

$I_{st.z} := 2 \cdot t_{ss} \cdot h_{ss} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$a_{webb} := \frac{b_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ww}}{2} = 0.525\,m$

$a_{webb} := a_{st.z.1} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$

$a_{webb} := a_{st.z.2} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$

Moment of inirtia of 15st of the web z-direction

$I_{stw.z} := 2 \cdot b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.491$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{sl} = 0$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4 \cdot t_s \cdot \varepsilon \cdot \sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.493$

SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)^3 \cdot h_w\right)^3}}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{webb}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{webb}} + 2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{webb}}} & \text{otherwise} \\ \alpha^2 & \end{cases} = 1.635 \times 10^3$

$\lambda_w := \max\left(\frac{h_w}{\left(37.4 \cdot t_w \cdot \varepsilon \cdot \sqrt{k_\tau}\right)}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

has to be performed for stiffened webs:

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ $\frac{t_w}{t_s} \leq 31 \cdot \frac{\varepsilon\left(t_s\right)}{\eta} \cdot \sqrt{k_{\tau 1}} =$ "Reduce"

SS-EN 1993-1-5 5.3 Table 5.1, stiff and stiffener.

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M3} := 1.00$

S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.457 \times 10^3\,kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.578$ Contribution from two web

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{pl.eff.neg} := a_{tp.eff.neg} - \frac{t_d}{2} = 424.644\,mm$

$a_{sl.eff.neg} := h_n - \frac{\eta_b}{2} - a_{tp.eff.neg} = 452.356\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{sl.eff.neg} \cdot \left|a_{pl.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M1}} + \frac{A_{sl.eff.neg} \cdot \left|a_{sl.eff.neg}\right| \cdot f_y\left(t_s\right)}{\gamma_{M0}} = 7.465 \times 10^3\,kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{sl} := a \cdot \left[0.25 + \frac{1.6 \cdot b_{fl} \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_s \cdot \left(t_s + b_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.623\,m$

$c_{sl} := a \cdot \left[0.25 + \frac{1.6 \cdot b_{fl} \cdot t_b^2 \cdot f_y\left(t_b\right)}{t_s \cdot \left(t_s + b_{wy}\right)^2 \cdot f_y\left(t_s\right)}\right] = 0.59\,m$

$V_{bf.Rd.red} := \frac{b_{fl} \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{sl} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y\left(t_b\right)}{c_{sl} \cdot \gamma_{M1} \cdot \gamma_n} = 294.676\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.red} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\,kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_s\right) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} = V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3\,kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.578$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.tot} := I_{y.sl.eff.tot} + I_{y.b.tot} = 9.131 \times 10^{-3}\,m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{ss.eff} \cdot t_d + n_{ss} \cdot s_{ss.eff} \cdot t_{d.eff} + n_{ss} \cdot t_{ss} \cdot t_{d.eff} + n_{ss} \cdot t_{ss} \cdot t_{d.eff} + A_b = 412.987\,cm^2$

$z_{tp.f.eff.neg} := \frac{s_{ss.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{ss} \cdot s_{ss.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + \ldots + n_{ss} \cdot t_{ss} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_n - \frac{\eta_b}{2}\right)}{A_{eff.f.tot}} = 344.064\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.tot}}{z_{tp.f.eff.neg}} = 0.027\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 9.421 \times 10^3\,kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.114 \times 10^4\,kN \cdot m$

Moment capacity check when $q_M = 78\,\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.992$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The reduced stress method

-Changed top plate stiffener thickness to 8 mm

Material

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ (335MPa) & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} := 1$

$E := 210GPa$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot GPa$

Geometry

Bottom plate

$t_b := 16mm$ $b_b := 1000mm$ $A_b := b_b t_b = 0.016m^2$

Top plate

$t_d := 10mm$ $b_d := 3500mm$ $A_d := b_d t_d = 0.035m^2$

Side plates

$t_s := 12mm$ $h_s := 389.2mm$ $A_s := h_s t_s = 4.67\times10^{-3}m^2$

Webs

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w t_w = 0.016m^2$

$h_{w.s} := 1250mm$ $h_{w.y} := 482.8mm$

Top stiffeners

$t_{sti} := 8mm$ $h_{sti} := 140mm$ $A_{sti} := h_{sti} t_{sti} = 1.12\times10^{-3}m^2$

$x_{sti} := 350mm$ $n_{sti} := 9$

Web stiffeners

$t_{wi} := 12mm$ $h_{wi} := 120mm$ $A_{wi} := h_{wi} t_{wi} = 1.44\times10^{-3}m^2$

$x_{wi} := 440mm$ $n_{wi} := 3$

Whole bridge

$L_{bridge} := 10m$ $h_{tb} := \frac{h_b}{2} + h_{w.y} + h_s + t_d = 0.89m$ $\gamma_{tb} := 2m$

$I_{tot} := 0.013901m^4$ From grasshopper

$Y_{tp} := 0.501324m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 61\frac{kN}{m^2}$

$Q := q\cdot b_d = 213.5\frac{kN}{m}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 8.647\times10^3 kN\cdot m$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 1.921\times10^3 kN$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\left(h_b + t_d - \frac{t_d}{2} - Y_{tp}\right) = 238.621\cdot MPa$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d b_d}{n_{sti}+2}\left(h_b + \frac{t_d}{2} - Y_{tp}\right) = 1.221\times10^{-3}m^3$

$\tau_{Ed.tp} := \frac{V_{Ed} S_{tp}}{I_{tot}\cdot min(t_b, t_{sti})} = 21.09\cdot MPa$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\left(h_b + t_d - Y_{tp}\right) = 235.512\cdot MPa$

$\sigma_{z.Ed.sp} := \frac{Q}{2 t_s} = 8.896\cdot MPa$

$S_{sp} := \left(h_b + t_d - Y_{tp}\right) t_s \left(\frac{h_b + t_d - Y_{tp}}{2}\right) + \frac{t_d b_d}{2}\left(h_b + \frac{t_d}{2} - Y_{tp}\right) = 9.13\times10^{-3}m^3$

$\tau_{Ed.sps} := \frac{V_{Ed} S_{sps}}{I_{tot} t_{wo}} = 24.717\cdot MPa$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.94\frac{MPa}{\frac{kN}{m^2}} = 52.579\cdot MPa$ From a linear stress analysis in Sofistik

$\sigma_{x.sps.max.ULS} := 79.1$

$\sigma_V := \sigma_{V.UNIT} q = 301.34\cdot MPa$

$\sigma_{x.sps} := \frac{t_d\left(t_d\right)}{2}\left(h_b + t_d - Y_{tp}\right) = 235.512\cdot MPa$

$\alpha_{ult} := \frac{\sigma_V}{\sigma_{x.sps}} = 1.178$

$\sigma_{cr.UNIT} := 146.05\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\sigma_{V.UNIT}}{\eta} = 2.394$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.701$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.978$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot {t_d}^2}{12\cdot\left(1-\nu^2\right)\cdot {b_d}^2} = 1.549\cdot MPa$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot\left(1-\nu^2\right)} = 3.205\times10^{-7}m^4$

$y_{tp.sl.tp} := \frac{A_d\frac{t_d}{2} + n_{wo}A_{wo}\left(t_d + \frac{h_{wo}}{2}\right)}{A_d + n_{wo}A_{wo}} = 21.77\cdot mm$

$I_{sl.tp} := \frac{b_d t_d^3}{12} + A_d\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + \ldots = 6.078\times10^{-7}mm^4$

$\ldots + n_{wo}\left[\frac{t_{wo}h_{wo}^3}{12} + A_{wo}\left(t_d + \frac{h_{wo}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 189.626$

$\delta_{tp} := \frac{n_{wo}\cdot A_{wo}}{A_d} = 0.288$

$\alpha_{tp} := max\left(\frac{h_{tb}}{b_d}, 0.5\right) = 0.571$ $\sqrt{\gamma_{tp}} = 3.711$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\left(\psi_{tp}+1\right)\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} > \sqrt{\gamma_{tp}} = 452.682 \\ \frac{4\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{tp}+1\right)\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} > \sqrt{\gamma_{tp}} \end{cases}$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 701.381\cdot MPa$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}} b_{eo} = 0.175m$

$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}} b_{eo} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35m$

$y_{tp.sl.1.tp} := \frac{t_d b_{sl.1.tp}\frac{t_d}{2} + A_{wo}\left(t_d + \frac{h_{wo}}{2}\right)}{t_d b_{sl.1.tp} + A_{wo}} = 0.023m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^3}{12} + b_{sl.1.tp} t_d\left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{wo}h_{wo}^3}{12} + A_{wo}\left(t_d + \frac{h_{wo}}{2} - y_{tp.sl.1.tp}\right)^2$ Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot {b_s}^2} = 743.723\cdot MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.038m$

$e_{tp} := max\left(y_{tp.sl.1.tp} - t_d - t_d + \frac{h_{wo}}{2} - y_{tp.sl.1.tp}\right) = 0.057m$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.625$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e.tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.903$ [6.3.1.2]

$\chi_{tp} := min\left(\frac{1}{\left(\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}\right)}, 1\right) = 0.68$

Interpolation

$\xi_{tp} := min\left(max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.68$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.183$

$\frac{\rho_{c.tp}\alpha_{ult}}{\gamma_{M1}} = 0.801$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}z + \sigma_{max}\right] 1$

$\psi_{sp.x} := \frac{\sigma\left(h_b - t_d - h_s\right)}{\sigma\left(h_b - t_d\right)} = -0.028$

$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\left(h_b - t_d - Y_{tp}\right) = 235.512\cdot MPa$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\left(h_b - t_d - h_s - Y_{tp}\right) = -6.545\cdot MPa$

$\psi_{sp1.x} := max\left(3 + \psi_{sp.x}, 0\right) = 2.972$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.028$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(\psi_{sp1.x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1.093$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} \ge 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp.x}\right)^2} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29\cdot\psi_{sp.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot\left(1 - \psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 7.992$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.442\times10^3 MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot {t_s}^2}{12\cdot\left(1-\nu^2\right)\cdot {h_s}^2} = 6.833\cdot MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.799$

$\chi_{sp} := min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)}, 1\right) = 0.847$

Interpolation

$\xi_{sp.x} := min\left(max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) - \right] & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) - \right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{sp} & \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) + \chi_{sp}\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} := 1$

$\sigma_{p.sp} := 0.34$ $\lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp}\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p^2\right] = 0.851$

$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.998$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot {t_s}^2}{12\cdot\left(1-\nu^2\right)\cdot {h_s}^2} = 6.833\cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 27.331\cdot MPa$

Column-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot {t_s}^2}{12\cdot\left(1-\nu^2\right)\cdot {b_s}^2} = 180.432\cdot MPa$

Interpolation

$\xi_{sp.z} := min\left(max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2-\xi_{sp.z}\right) - \right] & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2-\xi_{sp.z}\right) - \right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{sp} & \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2-\xi_{sp.z}\right) + \chi_{sp}\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.847$

Top plate stiffeners - check

x-direction

$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$$\psi_{sp,x} := \frac{\sigma(h_n - t_f - h_{wb})}{\sigma(h_n - t_f)} = 0.63$$

$$\sigma_{E,sp,x} := \frac{\pi^2 \cdot E \cdot t_{wb}^2}{12\left(1 - \nu^2\right) \cdot h_{wb}^2} = 619.755 \cdot \text{MPa}$$

$$k_{\sigma,sp,x} := \begin{cases} 1 & \text{if } \psi_{sp,x} \equiv 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{sp,x}\right)} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} \equiv 0 \\ 7.81 - 6.29 \cdot \psi_{sp,x} + 9.78 \cdot \psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} \equiv -1 \\ 5.98 \cdot \left(1 - \psi_{sp,x}\right)^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 85.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 3.853$$

$$\sigma_{cr,p,sp,x} := k_{\sigma,sp,x} \cdot \sigma_{E,sp,x} = 2.388 \times 10^3 \cdot \text{MPa}$$

Column-like behaviour

$$\sigma_{cr,c,sp,x} := \frac{\pi^2 \cdot E \cdot i_{sa}^2}{12\left(1 - \nu^2\right) \cdot a_{sa}^2} = 3.037 \cdot \text{MPa}$$

$$\chi_{sp,x} := \chi_{sp}$$

Interpolation

$$\xi_{sp,x} := \min\left(\max\left(\frac{\sigma_{cr,p,sp,x}}{\sigma_{cr,c,sp,x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c,sp,x} := \begin{cases} \left[\left(\beta_{sp,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2 - \xi_{sp,x}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\beta_{sp,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2 - \xi_{sp,x}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\beta_{sp,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2 - \xi_{sp,x}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y(t_f)}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y(t_f)}{\gamma_{M1}}}\right)^2 = 0.985$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y(t_w)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y(t_w)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y(t_w)}{\gamma_{M1}}}\right)\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y(t_w)}{\gamma_{M1}}}\right) \ldots = 0.468$$
$$+ 3\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y(t_w)}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.sps}}{\frac{\rho_{c,sps,x} \cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed.sps}}{\frac{\chi_w \cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.451$$

EWM - Changed top plate stiffener thickness

This chapter covers the EWM when changing the top plate stiffener thickness to 10 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s,so} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d,0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s,so}} = 350\ mm$

Side plate: $b_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longiudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 10mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939\ mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890\ mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffners: $a := 2m$

Transverse stiffners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864\ mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

$A_d := t_d \cdot b_d$ $A_s := 2 \cdot t_s \cdot b_s$ $A_w := 2 \cdot t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$ $A_{sl} := n_{sl} \cdot t_{sl} \cdot h_{sl}$

Material parameters:

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.114\ m^2$

$A_c := A_{tot} + A_d = 0.048\ m^2$ EC1993-1-1 tab. 3.1

Change of center of gravity z_{tp}

$\Delta(i) := \begin{cases} 1 & \text{if } i = 0 \\ \frac{s_{su0}}{t_d(i)} & \text{otherwise} \end{cases}$

$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{su} \cdot \left(t_d + \frac{h_{su}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots}{+ A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{sl} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\Delta(1)\right) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 381.913\ mm$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206\ mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.169 \times 10^{-3}\ m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 8.234 \times 10^{-4}\ m^4$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 \cdot s_w \cdot h_w \cdot \left[h_w - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 2.776 \times 10^{-3}\ m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.973 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_w - z_{tp} - \frac{t_b}{2}\right)^2 = 4.002 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s} := 2 \cdot \frac{t_s \cdot h_s^3}{12} + 2 \cdot t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 4.116 \times 10^{-4}\ m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.014\ m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\ MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(b_w, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max(3 + \psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\ mm$

$z_2 := t_d + h_s = 399.2\ mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.046$

$Condition := \begin{cases} 62 \cdot \varepsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$

$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$k_{\sigma.4.1}(\psi_s) = 8.124$

$s_{c.so} := s_{so} - s_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.492$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.017\ m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 371.913\ mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 371.913\ mm$

$h_{s.eff.c1} := 0.4\, h_{s.eff.c} = 148.765\ mm$

$h_{s.eff.c2} := 0.6\, h_{s.eff.c} = 223.148\ mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\ mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.926 \times 10^{-3}\ mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right) \cdot t_s = 9.341 \times 10^{-3}\ mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12\left(1 - \nu^2\right) h_s^2} = 1.466 \times 10^3\ MPa$ EC1993-1-5 eq. A.1

Stiffeners

$\Delta h_s := t_d = 10\ mm$

$h_{sw} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.624$

$k_{\sigma.4.2}(\psi_{so}) = 0.6$ Higher stress close to support

$Class4_{so} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \epsilon \cdot (k_{\sigma.4.2}(\psi_{so}))^{0.5} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2}(\psi_{so}), b_{so}, t_{so}) = 0.782$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_s) = 0.971$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so} \cdot b_{so} = 135.954\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot b_{so.eff} = 1.224 \times 10^{-4}\ m^2$

Deck

$\frac{b_{sw}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{sw} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{sw}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_{sd} := \frac{-t_d}{2} = -5 \times 10^{-3}\ m \qquad \psi_{sd} := \psi_{neg}(z_d, z_d) = 1$

$b_{sw} := h_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.sd} := k_{\sigma.4.1}(\psi_{sd}) = 4$

$\lambda_{p.sd} := \lambda_p(b_{sw.sd}, t_d, \epsilon, k_{\sigma.sd}) = 0.736$

$\rho_{sd} := \rho_{4.2}(\lambda_{p.sd}, \psi_{sd}) = 0.953$ Internal cross-sectional parts

$b_{sw.eff} := \rho_{sd} \cdot b_{sw} = 323.943\ mm$

Areas:

$A_{sw} := n_{sw} \cdot t_{sw} \cdot t_d + n_{so} \cdot t_{so} \cdot (b_{so.eff} + t_d) = 4.464 \times 10^{-4}\ m^2$

$A_{sw.eff.loc} := n_{sw} \cdot t_{sw.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (b_{so.eff} + t_d) = 4.229 \times 10^{-4}\ m^2$

More than three longitudinal stiffeners -> $k_{\sigma.sp}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5}\ m^4$

$y_{sp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + b_d \cdot t_d} = 24.853\ mm$

$I_d := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl}\right)^2\right] \dots = 7.299 \times 10^{-7}\ m^4$
$+ \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl}\right)^2\right]$

$\gamma := \frac{l_d}{l_p} = 227.715 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.013\ m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035\ m^2$

$\beta_A := \frac{A_{sl}}{A_p} = 0.36$

$k := \begin{cases} \left[2 \cdot \left[(1 + \alpha^2)^2 + \gamma - 1\right]\right] & \text{if } \alpha \geq \sqrt{\gamma} \\ \left[\left(\frac{2}{\alpha} + \alpha\right)^2 \cdot \gamma\right] & \text{otherwise} \end{cases} = 514.488$

$\lambda_{cr.p} := \sqrt{\frac{(\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.65 \quad \delta)$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour
Linear stressing versus, orthotropic plate:

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d = 4.553 \times 10^{-4}\ m^2$ 3-1-5 eq A.1

Buckling resistance deck

$\chi_{c.c} := \frac{A_{c.eff.loc}}{A_c} = 0.947$

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.616 \times 10^3\ kN$

Column like behaviour

$y_{sp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + n_{so} \cdot t_d} = 26.429\ mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl.c}\right)^2\right] \dots = 7.941 \times 10^{-6}\ m^4$
$+ \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl.c}\right)^2\right]$

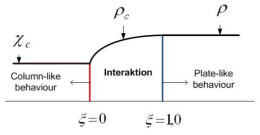
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + n_{so} \cdot t_d = 4.9 \times 10^{-3}\ m^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + n_{so.eff} \cdot t_d = 4.599 \times 10^{-3}\ m^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.939$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 839.709\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.63$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 122.045\ mm$

$e_c := \max\left(\left[\left(y_{sp.sl} - \frac{t_d}{2}\right) \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl}\right)\right]\right) = 55.147\ mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.531$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.812$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.754$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.754$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot t_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.377 \times 10^{-4}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot t_{so.eff} \cdot t_d + A_{so}} = -329.883\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{so})}{\gamma_M} = 267.802\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{so})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_{so} \cdot t_{so.eff} = 0.032\ m^2$

$A_{tot.eff} := A_{sw.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{s.eff} = 0.111\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} & A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ & + 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ & \left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \dots \\ & + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - x_{so0}) - \frac{h_{so.y}}{2}\right] \dots \\ & + A_b \cdot \left(h_{tb} - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 391.548\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right.$
$+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots$
$\left. + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] \dots + 4.444 \times 10^{-4}\ m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.841 \times 10^{-3}\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.014\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{bl.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 4.229 \times 10^{-4}\ m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_{so}) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -3.978 \times 10^{-3}\ m^2$

$A_{sl.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{comloc} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + n_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 351.424\ cm^2$

A_{st} Lower stiffener in tension -> no reduction

changed with reader area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \text{otherwise} \end{cases} = 7.544\ mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} & n_{so} \cdot h_{so.eff} \cdot s_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ & + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(t_d + \frac{t_{d.eff}}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ & + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ & + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - x_{so0}) - \frac{h_{so.y}}{2}\right] \dots \\ & + A_b \cdot \left(h_{tb} - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 416.309\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.481 \times 10^{-3}\ m^4$

$h_{stiff} := \sin\left(21.12 \frac{m}{180}\right) \cdot s_{so0} = 7.206\ mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.419 \times 10^{-3}\ m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots = 6.681 \times 10^{-4}\ m^4$
$+ 2 \cdot t_{so} \cdot h_{so} \cdot \left[t_d + h_s + \frac{1}{3} \cdot (h_{wy} - x_{so0y}) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$
$+ 2 \cdot t_{so} \cdot h_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - x_{so0y}) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$
$+ 2 \cdot t_{so} \cdot h_{so} \cdot \left[t_d + h_s + (h_{wy} - x_{so0y}) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\ m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{tb} - \frac{h_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp.eff.neg}\right]^2 = 2.242 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot h_b^3}{12} + t_b \cdot b_b \cdot \left(h_{tb} - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 3.47 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right.$
$+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots$
$\left. + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 5.366 \times 10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.382 \times 10^{-2}\ m^4$

Load

$q_M := 83 \frac{kN}{m^2}$

$Q_M := q_M \cdot b_d = 290.5 \frac{kN}{m} \qquad V_{Ed} := \frac{Q_M \cdot l_{bridge}}{2} = 2.615 \times 10^3\ kN$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.177 \times 10^4\ kN \cdot m$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.035\ m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma M1} = 1.178 \times 10^4\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.999$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$

$\alpha_w := asin\left(\frac{h_w}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_h}{2} + s_{wall} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{st}}{2} + \frac{sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{st}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{st} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{st} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$

Moment of inirta stiffeners z-direction

$I_{st.z} := 2 \cdot t_{st} \cdot h_{st} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15ct each side of the stiffeners:

$\overline{a_{web.1}} := \frac{b_h}{2} + s_{wall} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{st}}{2} = 0.525\,m$

$\overline{a_{web.2}} := a_{st.z.1} + s_{st} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$

$\overline{a_{web.3}} := a_{st.z.2} + s_{st} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$

Moment of inirtia of 15ct of the web z-direction

$I_{stw.z} := 2 b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00\left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34\left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.491$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq 5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\overline{\alpha_w} := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := max\left[\left(9 \cdot \left(\frac{h_w}{a}\right)^2 \cdot \sqrt{\frac{I_{st.z.tot}}{\left[max(t_f, t_w)\right]^3 \cdot h_w}}\right), \frac{2.1}{max(t_f, t_w)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^{3}$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{min(t_f, t_w)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{min(t_f, t_w)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^{3}$

$\lambda_w := max\left(\frac{h_w}{\left(37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_\tau}\right)}, \lambda_{w1}\right) = 0.455$, has to be per SS-EN 1993-1-5 5.3 (5.6)

$\overline{\gamma_M} := 1.2$ EC1993-1-3-5 ANM.2 5.1

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ SS-EN 1993-1-5 5.3 Table 5.1, stiff end stiffener.

$\frac{h_w}{t_w} \leq 31 \cdot \frac{\varepsilon(t_f)}{\eta} \cdot \sqrt{k_\tau} =$ "Reduce"

EC1003-1-5 Ch. 5.1(2).

Kapacity of the web

$\overline{\gamma_{M0}} := 1.00$ $\overline{\gamma_{M1}} := 1.2$ Safety class 3

$\overline{\gamma_{M0}} := 1.00$

SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_f) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^{3}\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.615 \times 10^{3}\,kN$

$\frac{V_{Ed}}{2 \cdot V_{bw.Rd}} = 0.615$ Contribution from two web

$V_{b.Rd} < \frac{\eta \cdot f_y(t_f) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{pl.eff.neg} := z_{pp.eff.neg} - \frac{t_f}{2} = 411.309\,mm$

$a_{pl.eff.neg} := h_h - \frac{t_h}{2} - z_{pp.eff.neg} = 465.691\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{pl.eff.neg} \cdot \left|a_{pl.eff.neg}\right| \cdot f_y(t_f)}{\gamma_{M0}} + \frac{A_{pl.eff.neg} \cdot \left|a_{pl.eff.neg}\right| \cdot f_y(t_h)}{\gamma_{M0}} = 7.375 \times 10^{3}\,kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{pl} := a \cdot \left[0.25 + \frac{1.6 \cdot b_fl \cdot t_f^2 \cdot f_y(t_f)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.623\,m$

$c_{pl} := a \cdot \left[0.25 + \frac{1.6 \cdot b_h \cdot t_h^2 \cdot f_y(t_h)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.59\,m$

$V_{bf.Rd.cred} := \frac{b_fl \cdot t_f^2 \cdot f_y(t_f)}{c_{pl} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_h \cdot t_h^2 \cdot f_y(t_h)}{c_{pl} \cdot \gamma_{M1} \cdot \gamma_n} = 294.676\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(3) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cred} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} = 0\,kN \\ 0 & \text{otherwise} \end{cases}$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} < \frac{\eta \cdot f_y(t_f) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$\overline{V_{Rd.tot}} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^{3}\,kN$

Check:

$\frac{V_{Ed}}{2} < 1 \cdot V_{b.Rd}$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 \cdot V_{b.Rd}} = 0.615$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.tot} := I_{y.d.eff.tot} + I_{y.h.tot} = 8.951 \times 10^{-3}\,m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{su} \cdot t_{su} \cdot t_{d.eff} + A_h = 419.12\,cm^2$

$z_{pp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \cdots + n_{su} \cdot t_{su} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_h \cdot \left(h_s - \frac{t_h}{2}\right)}{A_{eff.f.tot}} = 339.132\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.tot}}{z_{pp.f.eff.neg}} = 0.026\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 9.37 \times 10^{3}\,kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.178 \times 10^{4}\,kN \cdot m$

Moment capacity check when $q_{-}M = 83\,\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.999$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The reduced stress method

-Changed top plate stiffener thickness to 10 mm

Material

$f_y(t) = \begin{cases} 355\,\text{MPa} & \text{if } t < 40\,\text{mm} \\ 335\,\text{MPa} & \text{if } t > 40\,\text{mm} \end{cases}$ $\gamma_{M1} = 1$

$E = 210\,\text{GPa}$

$\nu = 0.3$ $G = \frac{E}{2\cdot(1+\nu)} = 80.769\,\text{GPa}$

Geometry

Bottom plate

$t_{bp} = 16\text{mm}$ $b_{bp} = 1000\text{mm}$ $A_{bp} = b_{bp} t_{bp} = 0.016\text{m}^2$

Top plate

$t_{tp} = 10\text{mm}$ $b_{tp} = 3500\text{mm}$ $A_{tp} = b_{tp} t_{tp} = 0.035\text{m}^2$

Side plates

$t_s = 12\text{mm}$ $h_s = 389.2\text{mm}$ $A_s = h_s t_s = 4.67 \cdot 10^{-3}\,\text{m}^2$

Webs

$t_w = 12\text{mm}$ $h_w = 1340\text{mm}$ $A_w = h_w t_w = 0.016\text{m}^2$

$h_{w,n} = 1258\text{mm}$ $h_{w,p} = 482.8\text{mm}$

Top stiffeners

$t_{st} = 10\text{mm}$ $b_{st} = 140\text{mm}$ $A_{st} = h_{st} t_{st} = 1.4 \cdot 10^{-3}\,\text{m}^2$

$s_{st} = 350\text{mm}$ $n_{st} = 9$

Web stiffeners

$t_{wst} = 12\text{mm}$ $h_{wst} = 120\text{mm}$ $A_{wst} = h_{wst} t_{wst} = 1.44 \cdot 10^{-3}\,\text{m}^2$

$s_{wst} = 440\text{mm}$ $n_{wst} = 3$

Whole bridge

$t_{buildup} = 18\text{in}$ $h_b = \frac{h_s}{2} + h_{w,p} + h_c + t_{tp} = 0.89\text{m}$ $s_b = 2\text{m}$

$I_{yy} = 0.01\,41\text{m}^4$ From grasshopper

$V_{cg} = 0.508\,158\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_{Ed} = 65\,\frac{\text{kN}}{\text{m}^2}$

$Q = q_{Ed} \cdot s_b = 227.5\,\frac{\text{kN}}{\text{m}}$

$M_{Ed} = \frac{Q \cdot s_{buildup}^2}{8} = 9.214 \cdot 10^3\,\text{kN m}$

$V_{Ed} = \frac{Q \cdot s_{buildup}}{2} = 2.047 \cdot 10^3\,\text{kN}$

Top plate

$\sigma_{x,Ed,tp} = \frac{M_{Ed}}{I_{yy}} \left(h_b - \frac{t_{tp}}{2} - y_{cg} \right) = 245.519\,\text{MPa}$ $\sigma_{z,Ed,tp} = 0$

$S_{tp} = \frac{t_{tp} b_{tp}}{n_{st}+2} \left(h_b - \frac{t_{tp}}{2} - y_{cg} \right) = 1.199 \cdot 10^{-3}\,\text{m}^3$

$\tau_{Ed,tp} = \frac{V_{Ed} \cdot S_{tp}}{t_w \cdot \min(t_{tp}, t_{st})} = 17.36\,\text{MPa}$

Side plates

$\sigma_{x,Ed,sp} = \frac{M_{Ed}}{I_{yy}} (h_b - t_{tp} - y_{cg}) = 242.261\,\text{MPa}$

$\sigma_{z,Ed,sp} = \frac{Q}{2 t_s} = 9.479\,\text{MPa}$

$S_{sp} = (h_b - t_{tp} - y_{cg}) t_s \frac{(h_b - t_{tp} - y_{cg})}{2} + \frac{t_{tp} b_{tp}}{n_{st}+2} \left(h_b - \frac{t_{tp}}{2} - y_{cg} \right) = 9.326 \cdot 10^{-3}\,\text{m}^3$

$S_{tps} = \left(b_{tp} \frac{t_{tp} b_{tp}}{n_{st}+2} + 2 \frac{t_s h_s^2}{2} \right) \left(h_b - t_{tp} - \frac{h_{bs}}{4} - y_{cg} \right) = 1.443 \cdot 10^{-3}\,\text{m}^3$

$- \frac{t_{tp} b_{tp}}{n_{st}+2} \left(h_b - \frac{t_{tp}}{2} - y_{cg} \right)$

Top plate stiffeners

$\tau_{Ed,tps} = \frac{V_{Ed} \cdot S_{tps}}{I_{yy} \cdot t_{st}} = 20.898\,\text{MPa} < 242.261\,\text{MPa}$

Governing slenderness from FE-software

$\sigma_{y,LSNE} = 4.8\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_{cr} = \sigma_{y,LSNE} \cdot \eta = 312\,\text{MPa}$

$\alpha_{ult} = \frac{f_y(t_p)}{\sigma_{cr}} = 1.138$

$\sigma_{cr,LSNE} = 162.64\,\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} = \frac{\sigma_{cr,LSNE}}{\eta} = 2.502$

$\lambda_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.674$ Slenderness for all parts

Top plate check

$\psi_{tp} = 1$

$\rho_{tp,tp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.999$

Plate-like behaviour [A.1]

$\sigma_{E,tp} = \frac{\pi^2 E t_{tp}^2}{12(1-\nu^2) b_{tp}^2} = 1.549\,\text{MPa}$

$I_{tp,tp} = \frac{b_{tp} t_{tp}^3}{12(1-\nu^2)} = 3.205 \cdot 10^{-7}\,\text{m}^4$

$z_{tp,tp,cg} = \frac{A_{tp} \frac{t_{tp}}{2} + n_{st} A_{st} \left(t_s + \frac{h_{bs}}{2} \right)}{A_{tp} + n_{st} A_{st}} = 24.853\,\text{mm}$

$I_{tp,tp} = \frac{b_{tp} t_{tp}^3}{12} + A_{tp} \left(z_{tp,tp,cg} - \frac{t_{tp}}{2} \right)^2 + \dots = 7.299 \cdot 10^{-7}\,\text{mm}^4$

$+ n_{st} \left[\frac{t_{st} h_{bs}^3}{12} + A_{st} \left(t_s + \frac{h_{bs}}{2} - z_{tp,tp,cg} \right)^2 \right]$

$\gamma_{tp} = \frac{I_{tp,tp}}{z_{tp,tp,cg}} = 227.715$

$k_{\sigma,tp} = \frac{b_{tp} t_{tp}}{A_{tp}} = 0.36$

$\alpha_{tp} = \max\left(\frac{z_{tp,tp,cg}}{t_{tp}}, 0.5 \right) = 0.571$

$k_{\sigma,tp,tp} = \begin{cases} \frac{2\left[(1+\alpha_{tp}^2)^2 \cdot \gamma_{tp} - 1 \right]}{\alpha_{tp}^2 (\gamma_{tp}-1)(1+\alpha_{tp})} & \text{if } \alpha_{tp} < \sqrt{\gamma_{tp}} \\ \frac{4(1+\sqrt{\gamma_{tp}})}{(\gamma_{tp}-1)(1+\alpha_{tp})} & \text{if } \alpha_{tp} > \sqrt{\gamma_{tp}} \end{cases} = 514.488$

$\sigma_{cr,tp,tp} = k_{\sigma,tp,tp} \cdot \sigma_{E,tp} = 797.142\,\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1,sol,tp} = \frac{3 \cdot s_{tp}}{5 - \psi_{tp}} = 0.175\text{m}$

$b_{2,sup,tp} = \frac{2}{5 - \psi_{tp}} s_{tp} = 0.175\text{m}$ Parts of the top plate that contributes to the stiffener

$b_{d,1,tp} = b_{1,sol,tp} + b_{2,sup,tp} = 0.35\text{m}$

$z_{tp,d,1,tp} = \frac{t_{tp} b_{d,1,tp} \frac{t_{tp}}{2} + A_{st} \left(t_s + \frac{h_{bs}}{2} \right)}{t_{tp} b_{d,1,tp} + A_{st}} = 0.026\text{m}$ Combined center of gravity for stiffener and contributing top plate.

$I_{d,1,tp} = \frac{b_{d,1,tp} t_{tp}^3}{12} + b_{d,1,tp} t_{tp} \left(z_{tp,d,1,tp} - \frac{t_{tp}}{2} \right)^2 + \frac{t_{st} h_{bs}^3}{12} + A_{st} \left(t_s + \frac{h_{bs}}{2} - z_{tp,d,1,tp} \right)^2$

Combined moment of inertia

$\sigma_{cr,d,tp} = \frac{\pi^2 E I_{d,1,tp}}{A_{d,1,tp} b_{st}^2} = 839.709\,\text{MPa}$ Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,tp} = \sigma_{cr,d}$

$\sigma_{cr,c,tp} = \sigma_{cr,d,tp}$

Reduction factor for column-like behaviour

$i_{tp} = \sqrt{\frac{I_{d,1,tp}}{A_{d,1,tp}}} = 0.04\text{m}$

$e_{tp} = \max\left(z_{tp,d,1,tp} - t_{tp}/2 + \frac{h_{bs}}{2} - z_{tp,d,1,tp}, 0 \right) = 0.054\text{m}$

$\alpha_{e,tp} = 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.61$ Stiffener with open cross section (curve c)

$\phi_{tp} = 0.5\left[1 + \alpha_{e,tp} (\lambda_p - 0.2) + \lambda_p^2 \right] = 0.872$ [6.3.1.2]

$\chi_{tp} = \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1 \right) = 0.702$

Interpolation

$t_{tp} = \min\left(\max\left(\frac{\sigma_{cr,tp,tp}}{\sigma_{cr,c,tp}} - 1.0, 0 \right), 1 \right) = 0$

$\rho_{c,tp} = \begin{cases} \left[(\rho_{tp,tp} - \chi_{tp}) b_{tp} (2 - t_{tp}) t_{tp} + \chi_{tp} \right] \frac{1}{b_{tp} t_{tp}} & \text{if } \frac{\left[(\rho_{tp,tp} - \chi_{tp}) b_{tp} (2 - t_{tp}) t_{tp} + \chi_{tp} \right] t_{st} h_{bs}}{I_{yy}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{tp,tp} - \chi_{tp}) b_{tp} (2 - t_{tp}) t_{tp} + \chi_{tp} \right] t_{st} h_{bs}}{I_{yy}} > 1 \end{cases} = 0.702$

Shear

$\eta = 1.2$ 7.1 (2)

$\lambda_w = \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} < \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p > 1.08 \end{cases} = 1.2$

$\frac{\rho_{c,tp} \cdot \sigma_{cr,tp}}{\gamma_{M1}} = 0.799$

Side plates check

x-direction

$\sigma_{max} = f_y(t_{sp})$

$\varepsilon(\sigma) = \left(\frac{-\sigma_{max}}{f(t_{sp})} + \varepsilon_{max} \right) - 1$

$\psi_{sp,x} = \frac{\varepsilon(h_b - t_s - h_c)}{\varepsilon(h_b - t_s)} = -0.047$

$\sigma_{x,sup} = \frac{M_{Ed}}{I_{yy}} (h_b - t_s - y_{cg}) = 242.261\,\text{MPa}$

$\sigma_{x,bot} = \frac{M_{Ed}}{I_{yy}} (h_b - t_s - h_c - y_{cg}) = -11.309\,\text{MPa}$

$\psi_{sp,x} = \max(3 \cdot \psi_{sp,x}, 0) = 2.953$ $\psi_{sp,1} = \frac{\sigma_{x,bot}}{\sigma_{x,sup}} = -0.047$

$\rho_{sp,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ 1 & \text{if } \psi_{sp,x} \geq 1 \\ \frac{8.2}{(1.05 + \psi_{sp,x})} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\,\psi_{sp,x} + 9.78\,\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98(1 - \psi_{sp,x})^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 95.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 1.126$

$k_{\sigma,sp,x} = 8.125$

$\sigma_{cr,sp,sp,x} = \sigma_{E,sp,x} k_{\sigma,sp,x} = 1.466 \cdot 10^3\,\text{MPa}$

Column-like behaviour

$\sigma_{cr,c,sp,x} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) b_{sp}^2} = 6.833\,\text{MPa}$

$\alpha_{sp} = 0.21$

$\phi_{sp} = 0.5\left[1 + \alpha_{sp} (\lambda_p - 0.2) + \lambda_p^2 \right] = 0.777$

$\chi_{sp} = \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1 \right) = 0.859$

Interpolation

$t_{sp,x} = \min\left(\max\left(\frac{\sigma_{cr,sp,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0, 0 \right), 1 \right) = 1$

$\rho_{c,sp,x} = \begin{cases} \left[(\rho_{sp,sp} - \chi_{sp}) b_{sp} (2 - t_{sp}) t_{sp} \right] \frac{1}{b_{sp} t_{sp}} & \text{if } \frac{\left[(\rho_{sp,sp} - \chi_{sp}) b_{sp} (2 - t_{sp}) t_{sp} \right] h_{bs}}{I_{yy}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{sp,sp} - \chi_{sp}) b_{sp} (2 - t_{sp}) t_{sp} \right] h_{bs}}{I_{yy}} > 1 \end{cases} = 1$

Z-direction

$\psi_{sp,z} = 1$

$\alpha_{sp,sp} = 0.34$ $\lambda_{p0,sp} = 0.7$

$\phi_{sp,sp} = 0.5\left[1 + \alpha_{sp,sp} (\lambda_p - \lambda_{p0,sp}) + \lambda_p^2 \right] = 0.833$

$\rho_{sp,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{sp,sp} + \sqrt{\phi_{sp,sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.029$

Plate-like behaviour

$k_{\sigma,sp,sp,z} = 4$

$\sigma_{E,sp,z} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) b_{sp}^2} = 6.833\,\text{MPa}$

$\sigma_{cr,sp,sp,z} = k_{\sigma,sp,sp,z} \sigma_{E,sp,z} = 27.331\,\text{MPa}$

Column-like behaviour

$\sigma_{cr,c,sp,z} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) b_{sp}^2} = 180.432\,\text{MPa}$

Interpolation

$t_{sp,z} = \min\left(\max\left(\frac{\sigma_{cr,sp,sp,z}}{\sigma_{cr,c,sp,z}} - 1.0, 0 \right), 1 \right) = 0$

$\rho_{c,sp,z} = \begin{cases} \left[(\rho_{sp,sp} - \chi_{sp}) b_{sp} (2 - t_{sp}) t_{sp} \right] \frac{1}{b_{sp} t_{sp}} & \text{if } \frac{\left[(\rho_{sp,sp} - \chi_{sp}) b_{sp} (2 - t_{sp}) t_{sp} \right] h_{bs}}{I_{yy}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{sp,sp} - \chi_{sp}) b_{sp} (2 - t_{sp}) t_{sp} \right] h_{bs}}{I_{yy}} > 1 \end{cases} = 0.859$

Top plate stiffeners - check

x-direction

$\rho_{sp,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$\psi_{sp,x} := \frac{\sigma_{sp,x1} \cdot (y_{sp,x} - y_{sp,x1}) - \sigma_{sp,x2} \cdot (y_{sp,x} - y_{sp,x2})}{\sigma_{sp,x1} \cdot (y_{sp,x} - y_{sp,x2})} = 0.623$

$\sigma_{E,sp,x} := \frac{\pi^2 \cdot E \cdot t_{sp,x}^2}{12 \cdot (1 - \nu^2) \cdot b_{sp,x}^2} = 968.368\ \text{MPa}$

$k_{sp,sp,x} := \begin{cases} 1 & \text{if } \psi_{sp,x} \neq 1 \\ \frac{8.2}{1.05 + \psi_{sp,x}} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp,x} + 9.78 \cdot \psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98 \cdot (1 - \psi_{sp,x})^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 95.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 3.91$

$\sigma_{cr,sp,sp,x} := k_{sp,sp,x} \cdot \sigma_{E,sp,x} = 3.786 \cdot 10^3\ \text{MPa}$

Column-like behaviour

$\sigma_{cr,c,sp,x} := \frac{\pi^2 \cdot E \cdot t_{sp}^2}{12 \cdot (1 - \nu^2) \cdot a_{sp}^2} = 4.745\ \text{MPa}$

$a_{sp,x} := a_{sp}$

Interpolation

$\xi_{sp,x} := \min\left(\max\left(\frac{\sigma_{cr,sp,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0\right), 1\right) = 1$

$\rho_{c,sp,x} := \begin{cases} \left[\left(\beta_{sp,sp} \cdot \lambda_{sp,x}\right) \cdot \left(2 - \xi_{sp,x}\right)\right]^{-1} & \text{if } \frac{\left[\left(\beta_{sp,sp} \cdot \lambda_{sp,x}\right) \cdot \left(2 - \xi_{sp,x}\right)\right]^{-1} \cdot \alpha_{sh}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\beta_{sp,sp} \cdot \lambda_{sp,x}\right) \cdot \left(2 - \xi_{sp,x}\right) \cdot \lambda_{sp}\right] \cdot \alpha_{sh}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Top plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.576$

Side plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 \cdot \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right) + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.498$

Top plate stiffeners check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.473$

EWM - Changed top plate stiffener thickness

This chapter covers the EWM when changing the top plate stiffener thickness to 12 mm

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s,so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s,so}} = 350\,mm$

Side plate: $b_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 12mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{ls} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d \cdot b_d$ $A_s := 2 t_s b_s$ $A_w := 2 t_w h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} t_{so} h_{so}$ $A_{su} := n_{su} t_{su} h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.116\,m^2$

$A_c := A_{so} + A_d = 0.05\,m^2$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \frac{\pi}{180}\right) s_{su0} = 7.206\,mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \frac{t_{so} h_{so}^3}{12} + n_{so} t_{so} h_{so} \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.344 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2 t_{su} h_{su} \left[\left(t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right)^2 \ldots = 8.552 \times 10^{-4}\,m^4\right.$
$+ 2 t_{su} h_{su} \left[\left(t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right)^2 \ldots\right.$
$\left. + 2 t_{su} h_{su} \left[\left(t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right)^2\right.\right.$

Web:

$I_{y.w} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w.1} := I_{y.w.1} + 2 t_w h_w \left[h_b - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 2.886 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.801 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b t_b^3}{12} + b_b t_b \left(h_b - z_{tp} - \frac{t_b}{2}\right)^2 = 4.107 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2 \frac{t_s b_s^3}{12} + 2 t_s b_s \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.903 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.014\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(b_w, b, t) := \frac{\frac{b_w}{t}}{28.4 \cdot \varepsilon(t) \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left(\frac{\lambda_p - 0.055\max(3 + \psi), 0}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2) Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.065$

$\text{Condition} := \begin{cases} 62.1\left(\lambda_s\right)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42.4\left(\lambda_s\right)}{0.67 + 0.33\,\psi_s} & \text{otherwise} \end{cases}$

$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$k_{\sigma.4.1}(\psi_s) = 8.262$

$s_{c.so} := s_{so} - s_{so}$ Table 5.2 ES1993-1-1

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), b_s, t_s) = 0.488$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.024\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 365.369\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s\left(h_{s.c}\right) = 365.369\,mm$

$h_{s.eff.c1} := 0.4\, h_{s.eff.c} = 146.147\,mm$

$h_{s.eff.c2} := 0.6\, h_{s.eff.c} = 219.221\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\,mm$

$A_{s.eff.c} := 2 h_{s.eff.c} t_s = 8.769 \times 10^{-3}\,mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right) t_s = 9.341 \times 10^{-3}\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \frac{E \cdot \pi^2 \cdot t_s^2}{12\left(1 - \nu^2\right) b_s^2} = 1.491 \times 10^3\,MPa$ EC1993-1-5 eq.A.1

Stiffeners

$\Delta t_d = t_d = 10\,mm$

$h_{sto} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{mg}(z_2)}{\sigma_{mg}(z_1)} = 0.617$

$k_{\sigma.4.2}(\psi_{so}) = 0.604$ Higher stress close to support

Class4 := "Class4" if $\frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon \cdot (k_{so}) \cdot \sqrt{k_{\sigma.4.2}(\psi_{so})}$ = "Lower than class 4"
"Lower than class 4" otherwise

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2}(\psi_{so}), h_{so}, t_{so}) = 0.65$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_u) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 1.512 \times 10^{-4}\ m^2$

Deck

$\frac{h_{so}}{t_d} = 35$ $42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ := "Class4" if $\frac{h_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4"
"Lower than class 4" otherwise

$z_d := -\frac{t_d}{2} = -5 \times 10^{-3}\ m$ $\psi_d := \psi_{mg}(z_d, \cdot t_d) = 1$

$k_{so} := n_{so} - 1_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, b_{so}, t_d) = 0.731$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.956$ Internal cross-sectional parts

$h_{so.eff} := \rho_d \cdot h_{so} = 323.125\ mm$

Areas:

$A_{so} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 4.77 \times 10^{-4}\ m^2$

$A_{c.eff.loc} := n_{so} \cdot t_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 4.528 \times 10^{-4}\ m^2$

More than three longitudinal stiffeners -> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5}\ mm^4$

$y_{tp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + b_d \cdot t_d} = 27.626\ mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \dots + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 8.438 \times 10^{-7}\ mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 263.266$ $\alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.015\ m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035\ m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.432$

$k_{\sigma.c} := \begin{cases} \frac{2 \cdot \left[(1 + \alpha^2)^2 \cdot \gamma - 1\right]}{\alpha^2} & \text{if } \alpha \le \sqrt{\gamma} \\ 0 & \text{otherwise} \end{cases} = 564.65$

$\lambda_{p.c} := \sqrt{\frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.621$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

orthotropic plate:

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + n_{so} \cdot t_{so} \cdot t_d = 4.851 \times 10^{-4}\ m^2$ 3-1-5 eq A.1

Buckling resistance:

$\chi_{A.c} := \frac{A_{c.eff}}{A_g} = 0.949$

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.722 \times 10^{4}\ kN$

Column like behaviour

$y_{tp.sl.c} := \frac{n_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot h_{so} + n_{so} \cdot t_d} = 29.324\ mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \dots + \left[\frac{n_{so} \cdot h_{so}^3}{12} + h_{so} \cdot n_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 9.158 \times 10^{-6}\ mm^4$

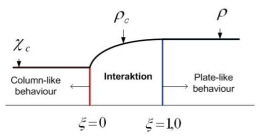
Buckling resistance stiffener

$A_{sl} := n_{so} \cdot h_{so} + n_{so} \cdot t_d = 5.18 \times 10^{-3}\ mm^2$

$A_{sl.eff} := n_{so} \cdot h_{so.eff} + n_{so.eff} \cdot t_d = 4.911 \times 10^{-3}\ mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.948$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 916.103\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{f_y(t_{so})}{\sigma_{cr.sl}}} = 0.606$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 127.631\ mm$

$z_c := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 52.374\ mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{z_c}} = 0.527$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.791$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.77$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero -> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.77$

Checks

Mean stress rpd

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{mg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{mg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.441 \times 10^{4}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -326.808\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{so})}{\gamma_M} = 273.401\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_d)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases}$ = "OK"

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{so} \cdot s_{so} \cdot t_{so.eff} = 0.032\ m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_{sl} + A_{so} + A_{fo} + A_b + A_{s.eff} = 0.114\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} & A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ & + 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ & \left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \dots \\ & + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{fo} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{so.fo}\right) - \frac{h_{so.y}}{2}\right] \dots \\ & + A_b \cdot \left(h_{ts} - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 384.133\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 4.19 \times 10^{-4}\ m^4$

$I_{y.d.eff} := \frac{n_{so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{so} \cdot s_{so} \cdot t_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.645 \times 10^{-3}\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{bf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 4.528 \times 10^{-4}\ m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.344 \times 10^{-3}\ m^2$

$A_{sl.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{bottle} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + n_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 381.043\ cm^2$

A_{so} Lower stiffener in tension -> no reduction

reduced evenly distributed over the cross section. Therefore the web-thickness is changed with ρ_c and the reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.701\ mm$

$\begin{aligned} & n_{so} \cdot s_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ & + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ & + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ & + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{so.fo}\right) - \frac{h_{so.y}}{2}\right] \dots \\ & + A_b \cdot \left(h_{ts} - \frac{h_b}{2}\right) \end{aligned}$

$z_{tp.eff.neg} := \frac{\quad}{A_{c.eff.tot}} = 406.352\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.205 \times 10^{-3}\ m^4$

$\alpha_{outer} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{so.fo} = 7.206\ mm$

Stiffeners overlunder:

$I_{y.st.so.fin} := n_{so} \cdot \frac{s_{so.eff} \cdot t_{so.eff}^3}{12} + n_{so} \cdot s_{so} \cdot t_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.635 \times 10^{-3}\ m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots + 2 \cdot s_{su} \cdot h_{su} \cdot \left[\left(t_d + h_s + \frac{1}{2} \cdot (h_{wy} - s_{so.fo})\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot s_{su} \cdot h_{su} \cdot \left[\left(t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{so.fo})\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot s_{su} \cdot h_{su} \cdot \left[\left(t_d + h_s + (h_{wy} - s_{so.fo})\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 = 7.11 \times 10^{-4}\ m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\ m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[\left(h_{ts} - \frac{h_b}{2} - \frac{1}{2} \cdot (h_{wy})\right) - z_{tp.eff.neg}\right]^2 = 2.389 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot h_b^3}{12} + t_b \cdot h_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 3.62 \times 10^{-7}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 4.981 \times 10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.406 \times 10^{-10}\ mm^4$

Load

$q_M := 86 \frac{kN}{m^2}$

$Q_M := q_M \cdot b_d = 301 \frac{kN}{m}$ $V_{Ed} := \frac{Q_M \cdot l_{bridge}}{2} = 2.709 \times 10^{3}\ kN$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.219 \times 10^{4}\ kN \cdot m$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.035\ m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.228 \times 10^{4}\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.993$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \epsilon(t_w) \cdot t_w = 0.293\,m$

$\alpha_w := \min\left(\frac{b_{wy}}{h_w}, \frac{180}{\pi}\right) = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{sw}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$

Moment of inrtia stiffeners z-direction (two stiffeners)

$I_{st.z} := 2 \cdot t_{sw} \cdot h_{sw} \cdot \left[(a_{st.z.1})^2 + (a_{st.z.2})^2 + (a_{st.z.3})^2\right] = 8.18 \times 10^{-3}\,m^4$

Distance from z-axis to middle of the web area which is 15t each side of the stiffeners:

$a_{web.1} := \frac{h_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} = 0.525\,m$

$a_{web.1} := a_{st.z.1} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$

$a_{web.2} := a_{st.z.2} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$

Moment of inrtia of 15st of the web z-direction

$I_{stw.z} := 2 b_{web} \cdot t_w^3 \cdot \left[(a_{st.z.1})^2 + (a_{st.z.2})^2 + (a_{st.z.3})^2\right] = 0.021\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00\left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34\left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.491$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4 \cdot t_w \cdot \epsilon(t_w) \cdot \sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau 2} := \max\left[\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min(t_f, t_w)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min(t_f, t_w)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^{-3}$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau 2} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18\frac{I_{st.z.tot}}{\min(t_f, t_w)^3 \cdot h_{web}} + 2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min(t_f, t_w)^3 \cdot h_{web}}} & \text{otherwise} \\ 4.1 + \frac{2}{\alpha^2}\sqrt[3]{\frac{I_{st.z.tot}}{\min(t_f, t_w)^3 \cdot h_{web}}} & \end{cases} = 1.635 \times 10^3$

$\lambda_w := \max\left(\frac{h_w}{37.4 \cdot t_w \cdot \epsilon(t_w) \cdot \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ has to be performed for stiffened webs SS-EN 1993-1-5 5.3 (5.6)

EC1993-1-5-5 ANM.2 5.1

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ $\frac{\eta}{\chi_w} \leq 31 \cdot \frac{\epsilon(t_w)}{\eta} \cdot \sqrt{\frac{a_{st}}{t_w}}$ SS-EN 1993-1-5 5.3 Table 5.1, but is satisfied.

EC1003-1-5 Ch. 5.1(2).

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.709 \times 10^3\,kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.637$ Contribution from two web

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{pl.eff.neg} := a_{tp.eff.neg} - \frac{t_d}{2} = 401.352\,mm$

$a_{sf.eff.neg} := h_b - \frac{t_b}{2} - a_{tp.eff.neg} = 475.648\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{sf.eff.neg} \cdot \left|a_{pl.eff.neg}\right| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{sf.eff.neg} \cdot \left|a_{sf.eff.neg}\right| \cdot f_y(t_b)}{\gamma_{M0}} = 7.306 \times 10^3\,kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{pl} := a \cdot \left[0.25 + \frac{1.6\,b_d \cdot t_d^2 \cdot f_y(t_d)}{t_f \cdot \left(h_w + h_{wy}\right)^2 \cdot f_y(t_f)}\right] = 0.623\,m$

$c_{sf} := a \cdot \left[0.25 + \frac{1.6\,b_b \cdot t_b^2 \cdot f_y(t_b)}{t_f \cdot \left(h_w + h_{wy}\right)^2 \cdot f_y(t_f)}\right] = 0.59\,m$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{pl} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{sf} \cdot \gamma_{M1} \cdot \gamma_n} = 294.676\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\,kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{wbf.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3\,kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.637$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.825 \times 10^{-3}\,m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{m.eff} \cdot t_d + n_{sw} \cdot s_{m.eff} \cdot t_{d.eff} + n_{sw} \cdot t_{sw} \cdot t_{d.eff} + A_b = 424.598\,cm^2$

$z_{tp.f.eff.neg} := \frac{s_{m.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{sw} \cdot s_{m.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \ldots + n_{sw} \cdot t_{sw} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_w - \frac{h_b}{2}\right)}{A_{eff.f.tot}} = 334.849\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.026\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 9.357 \times 10^3\,kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.228 \times 10^4\,kN \cdot m$

Moment capacity check when $q_{-}M = 80\,\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.993$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The reduced stress method

-Changed top plate stiffener thickness to 12 mm

Material

$f_{y0} := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ 335\,MPa & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$E = 210\,GPa$

$\nu = 0.3$ $G = \frac{E}{2\,(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate:

$t_b = 16\,mm$ $b_b = 1000\,mm$ $A_b = b_b t_b = 0.016\,m^2$

Top plate

$t_{tp} = 10\,mm$ $b_{tp} = 3500\,mm$ $A_{tp} = b_{tp} t_{tp} = 0.035\,m^2$

Side plates

$t_s = 12\,mm$ $h_s = 389.2\,mm$ $A_s = h_s t_s = 4.67 \times 10^{-3}\,m^2$

Webs:

$t_w = 12\,mm$ $h_w = 1340\,mm$ $A_w = h_w t_w = 0.016\,m^2$

$h_{w,k} = 1258\,mm$ $h_{w,fl} = 482.8\,mm$

Top stiffeners

$t_{st} = 12\,mm$ $b_{st} = 140\,mm$ $A_{st} = b_{st} t_{st} = 1.68 \times 10^{-3}\,m^2$

$s_{st} = 350\,mm$ $n_{st} = 9$

Web stiffeners

$t_{st} = 12\,mm$ $b_{st} = 120\,mm$ $A_{st} = b_{st} t_{st} = 1.44 \times 10^{-3}\,m^2$

$s_{st} = 440\,mm$ $n_{st} = 3$

Whole bridge

$L_{bridge} = 18\,m$ $b_b = \frac{b_b}{2} + b_{wg} + b_s + t_d = 0.89\,m$ $n_b = 2\,m$

$I_{yy} = 0.01437\,m^4$ From grasshopper

$y_{gt} = 0.514695\,m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_{-} = 70.4\,\frac{kN}{m}$

$Q = q_{-} n_b = 245\,\frac{kN}{m}$

$M_{Ed} = \frac{Q\,L_{bridge}^2}{8} = 9.922 \times 10^3\,kN\,m$

$V_{Ed} = \frac{Q\,L_{bridge}}{2} = 2.205 \times 10^3\,kN$

Top plate:

$\sigma_{x,Ed,tp} = \frac{M_{Ed}}{I_{yy}} \left(h_w - y_{gt} - \frac{t_{tp}}{2} - y_{tp} \right) = 255.696\,MPa$ $\sigma_{z,Ed,tp} = 0$

$S_{tp} = \frac{t_{tp} b_{tp}}{n_{bri} - 1} \left(h_w - y_{gt} - \frac{t_{tp}}{2} - y_{tp} \right) = 1.178 \times 10^{-3}\,m^3$

$\tau_{Ed,tp} = \frac{V_{Ed}\,S_{tp}}{t_{tp}\,\sin(\gamma_{tp})\,I_{yy}} = 15.066\,MPa$

Side plates:

$\sigma_{x,Ed,sp} = \frac{M_{Ed}}{I_{yy}} \left(h_w - y_{gt} - y_{sp} \right) = 252.243\,MPa$

$\sigma_{z,Ed,sp} = \frac{Q}{n_b} = 10.208\,MPa$

$S_{sp} = \left(h_w - y_{gt} - y_{sp} \right) t_s \frac{\left(h_w - y_{gt} - y_{sp} \right)}{2} + \frac{t_{tp} b_{tp}}{n_{bri} - 1} \left(h_w - y_{gt} - \frac{t_{tp}}{2} - y_{sp} \right) = 9.514 \times 10^{-3}\,m^3$

$S_{tps} = \begin{pmatrix} \frac{t_{tp} b_{tp}}{n_{bri} - 1} \\ \left(t_{tp} \frac{t_{tp} b_{tp}}{n_{bri} - 1} + 2\,t_s h_s \right) \frac{1}{2} \\ \left(h_s - y_{gt} - \frac{h_{tp}}{4} - y_{sp} \right) \\ + 2\,t_s h_s \left(h_s - y_{gt} - \frac{t_{tp}}{2} - y_{sp} \right) \end{pmatrix} = 1.456 \times 10^{-3}\,m^3$

Tim nlato stiffeners:

$\tau_{Ed,tp,s} = \frac{V_{Ed}\,S_{tps}}{t_{tp}\,I_{yy}} = 18.614\,MPa$

$\sigma_{x,Ed,tps} = \frac{M_{Ed}}{I_{yy}} \left(h_w - y_{gt} - y_{sp} \right) = 252.243\,MPa$

Governing slenderness from FE-software

$\sigma_{y,1,UNIT} = 4.69\,\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

$\sigma_y = \sigma_{y,1,UNIT} q_{-} = 327.6\,MPa$

$\alpha_{ult} = \frac{f_y(t_s)}{\sigma_y} = 1.084$

$\alpha_{cr,1,UNIT} = 182.67\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistk

$\alpha_{cr} = \frac{\alpha_{cr,1,UNIT}}{q_{-}} = 2.61$

$\lambda_y = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.644$ Slenderness for all parts

Top plate check

$\alpha_{tp} = 1$

$\rho_{p,tp} = \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\,(3 + \psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E,tp} = \frac{\pi^2 E t_{tp}^2}{12\,(1-\nu^2)\,b_{tp}^2} = 1.549\,MPa$

$I_{tp} = \frac{b_{tp} t_{tp}^3}{12\,(1-\nu^2)} = 3.205 \times 10^{-7}\,m^4$

$z_{tp,sl,tp} = \frac{A_{tp} \frac{t_{tp}}{2} + n_{st} A_{st} \left(t_d + \frac{h_{st}}{2} \right)}{A_{tp} + n_{st} A_{st}} = 27.626\,mm$

$I_{sl,tp} = \frac{b_{tp} t_{tp}^3}{12} + A_{tp} \left(z_{tp,sl,tp} - \frac{t_{tp}}{2} \right)^2 + \dots = 8.438 \times 10^{-7}\,mm^4$
$+ n_{st} \left[\frac{t_{st} h_{st}^3}{12} + A_{st} \left(t_d + \frac{h_{st}}{2} - z_{tp,sl,tp} \right)^2 \right]$

$\gamma_{sl} = \frac{I_{sl,tp}}{I_{tp,tp}} = 263.266$

$\delta_{tp} = \frac{n_{st} A_{st}}{A_{tp}} = 0.432$

$\alpha_{sl} = \max \left(\frac{\gamma_{sl}}{\gamma_{sl}}, 0.5 \right) = 0.571$

$k_{\sigma,tp,tp} = \begin{cases} \frac{2 \left[(1 + \alpha_{sl}^2)^2 + \gamma_{sl} - 1 \right]}{\alpha_{sl}^2 (\alpha_{sl} - 1)(1 + \delta_{tp})} & \text{if } \alpha_{sl} < \sqrt{\gamma_{sl}} \\ \frac{4 (1 + \sqrt{\gamma_{sl}})}{(\alpha_{sl} + 1)(1 + \delta_{tp})} & \text{if } \alpha_{sl} \ge \sqrt{\gamma_{sl}} \end{cases} = 564.65$

$\sigma_{cr,tp,tp} = k_{\sigma,tp,tp} \sigma_{E,tp} = 874.862\,MPa$

Column-like behaviour [4.5.3]

$b_{1,tot,tp} = \frac{3 - \psi_{tp}}{5 - \psi_{tp}} b_{tp} = 0.175\,m$

$b_{2,sup,tp} = \frac{2}{5 - \psi_{tp}} b_{tp} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{sl,1,tp} = b_{1,tot,tp} + b_{2,sup,tp} = 0.35\,m$

$z_{tp,sl,1,tp} = \frac{t_d t_{sl,1,tp} \frac{t_d}{2} + A_{tp} \left(t_d + \frac{t_{tp}}{2} \right) + \dots}{t_d t_{sl,1,tp} + A_{tp}} = 0.029\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl,1,tp} = \frac{b_{sl,1,tp} t_d^3}{12} + b_{sl,1,tp} t_d \left(z_{tp,sl,1,tp} - \frac{t_d}{2} \right)^2 + \frac{t_{tp} b_{tp}^3}{12} + A_{tp} \left(t_d + \frac{b_{tp}}{2} - z_{tp,sl,1,tp} \right)^2$

Combined moment of inertia

$\sigma_{cr,sl,tp} = \frac{\pi^2 E I_{sl,1,tp}}{A_{sl,1,tp}\,b_{tp}^2} = 916.103\,MPa$

Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,tp} = \sigma_{cr,sl}$

$\sigma_{cr,c,tp} = \sigma_{cr,sl,tp}$

Reduction factor for column-like behaviour

$i_{tp} = \sqrt{\frac{I_{sl,1,tp}}{A_{sl,1,tp}}} = 0.042\,m$

$e_{tp} = \max \left(z_{tp,sl,1,tp} - y_{tp} t_d + \frac{b_{tp}}{2} - z_{tp,sl,1,tp}, 0 \right) = 0.051\,m$

$\alpha_{e,tp} = 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.598$ Stiffener with open cross section (curve c)

$\phi_{tp} = 0.5 \left[1 + \alpha_{e,tp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 0.841$ [6.3.1.2]

$\chi_{tp} = \min \left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1 \right) = 0.724$

Interpolation

$t_{tp} = \min \left(\max \left(\frac{\sigma_{cr,tp,tp}}{\sigma_{cr,c,tp}} - 1.0, 0 \right), 1 \right) = 0$

$\sigma_{c,tp} = \begin{cases} \left[\left[\rho_{p,tp} - \chi_{tp} \right] b_{tp} \left(2 - t_{tp} \right) - t_d^2 \right] \frac{\gamma_{M0}}{\gamma_{M1}} & \text{if } \frac{\left[\left[\rho_{p,tp} - \chi_{tp} \right] b_{tp} \left(2 - t_{tp} \right) - t_d^2 \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left[\rho_{p,tp} - \chi_{tp} \right] b_{tp} \left(2 - t_{tp} \right) - t_d^2 \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.724$

Shear

$\eta = 1.2$ 7.1 (2)

$\lambda_w = \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$

$\frac{\sigma_{c,tp} \gamma_{M0}}{\gamma_{M1}} = 0.785$

Side plates check

x-direction

$\alpha_{max} = f_y(t_d)$

$\sigma(z) = \left[\frac{-\sigma_{max}}{f_y(t_d)} + 1 \right] \sigma_{max}$ 1

$\psi_{sp,x} = \frac{\sigma(h_s - y_{gt} - t_d)}{\sigma(h_s - y_{gt})} = -0.065$

$\sigma_{x,sp} = \frac{M_{Ed}}{I_{yy}} \left(h_w - y_{gt} - y_{sp} \right) = 252.243\,MPa$

$\sigma_{x,bot} = \frac{M_{Ed}}{I_{yy}} \left(h_w - y_{gt} - h_s - y_{sp} \right) = -16.5\,MPa$

$\psi_{sp3,x} = \max(3 + \psi_{sp,x}, 0) = 2.935$ $\psi_{sp3} = \frac{\sigma_{x,bot}}{\sigma_{x,sp}} = -0.065$

$\rho_{p,sp,sp} = \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\,(3 + \psi_{sp3})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

$k_{\sigma,sp,x} = \begin{cases} 1 & \text{if } \psi_{sp,x} \ge 1 \\ \frac{8.2}{(1.05 + \psi_{sp,x})} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\,\psi_{sp,x} + 9.78\,\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98\,(1 - \psi_{sp,x})^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 95.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 8.263$

$\sigma_{cr,sp,sp,x} = \sigma_{E,sp,x} k_{\sigma,sp,x} = 1.491 \times 10^3\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,x} = \frac{\pi^2 E t_s^2}{12\,(1-\nu^2)\,b_{sp}^2} = 6.833\,MPa$

$\alpha_{sp} = 0.21$

$\phi_{sp} = 0.5 \left[1 + \alpha_{sp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 0.754$

$\chi_{sp} = \min \left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1 \right) = 0.872$

Interpolation

$t_{sp,x} = \min \left(\max \left(\frac{\sigma_{cr,sp,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0, 0 \right), 1 \right) = 1$

$\sigma_{c,sp,x} = \begin{cases} \left[\left[\rho_{p,sp} - \chi_{sp} \right] b_{sp} \left(2 - t_{sp} \right) - t_d^2 \right] \frac{\gamma_{M0}}{\gamma_{M1}} & \text{if } \frac{\left[\left[\rho_{p,sp} - \chi_{sp} \right] b_{sp} \left(2 - t_{sp} \right) - t_d^2 \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left[\rho_{p,sp} - \chi_{sp} \right] b_{sp} \left(2 - t_{sp} \right) - t_d^2 \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

z-direction

$\psi_{sp,z} = 1$

$\alpha_{p,sp} = 0.34$ $\lambda_{p0,sp} = 0.7$

$\phi_{p,sp} = 0.5 \left[1 + \alpha_{p,sp} \left(\lambda_p - \lambda_{p0,sp} \right) + \lambda_p^2 \right] = 0.813$

$\rho_{p,z,sp} = \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\phi_{p,sp} + \sqrt{\phi_{p,sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,p,sp,z} = 4$

$\sigma_{E,sp,z} = \frac{\pi^2 E t_s^2}{12\,(1-\nu^2)\,b_{sp}^2} = 6.833\,MPa$

$\sigma_{cr,p,sp,z} = k_{\sigma,p,sp,z} \sigma_{E,sp,z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,z} = \frac{\pi^2 E t_s^2}{12\,(1-\nu^2)\,h_s^2} = 180.432\,MPa$

Interpolation

$t_{sp,z} = \min \left(\max \left(\frac{\sigma_{cr,p,sp,z}}{\sigma_{cr,c,sp,z}} - 1.0, 0 \right), 1 \right) = 0$

$\sigma_{c,sp,z} = \begin{cases} \left[\left[\rho_{p,sp} - \chi_{sp} \right] b_{sp} \left(2 - t_{sp} \right) - t_d^2 \right] \frac{\gamma_{M0}}{\gamma_{M1}} & \text{if } \frac{\left[\left[\rho_{p,sp} - \chi_{sp} \right] b_{sp} \left(2 - t_{sp} \right) - t_d^2 \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left[\rho_{p,sp} - \chi_{sp} \right] b_{sp} \left(2 - t_{sp} \right) - t_d^2 \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.872$

Top plate stiffeners - check

x-direction

$\rho_{p,x,tps} = \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$\psi_{sp,x} = \frac{\sigma(h_w - y_{gt} - h_{sl})}{\sigma(h_w - y_{gt})} = 0.617$

$$\sigma_{E,cpt,x} := \frac{\pi^2 \cdot E \cdot t_{sp}^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_{sp}^2} = 1.394 \times 10^3 \cdot \text{MPa}$$

$$k_{\sigma,cpt,x} := \begin{cases} 1 & \text{if } \psi_{cpt,x} \geq 1 \\ \frac{8.2}{1.05 + \psi_{cpt,x}} & \text{if } 1 > \psi_{cpt,x} > 0 \\ 7.81 & \text{if } \psi_{cpt,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{cpt,x} + 9.78 \cdot \psi_{cpt,x}^2 & \text{if } 0 > \psi_{cpt,x} > -1 \\ 23.9 & \text{if } \psi_{cpt,x} = -1 \\ 5.98 \cdot \left(1 - \psi_{cpt,x}\right)^2 & \text{if } -1 > \psi_{cpt,x} > -3 \\ 95.68 & \text{if } \psi_{cpt,x} < -3 \end{cases} = 3.972$$

$$\sigma_{cr,cpt,x} := k_{\sigma,cpt,x} \cdot \sigma_{E,cpt,x} = 5.539 \times 10^3 \cdot \text{MPa}$$

Column-like behaviour

$$\sigma_{cr,c,cpt,x} := \frac{\pi^2 \cdot E \cdot t_{sp}^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_{sp}^2} = 6.833 \cdot \text{MPa}$$

$$\chi_{cpt} := \chi_{sp}$$

Interpolation

$$\xi_{cpt,x} := \min\left(\max\left(\frac{\sigma_{cr,cpt,x}}{\sigma_{cr,c,cpt,x}} - 1.0, 1\right), 1\right) = 1$$

$$\rho_{c,cpt,x} := \begin{cases} \left[\left(\rho_{pl,cpt} - \chi_{sp}\right) \cdot b_{cpt,x} \cdot \left(2 - b_{cpt,x}\right) - \left[\left(\rho_{pl,cpt} - \chi_{sp}\right) \cdot b_{cpt,x} \cdot \left(2 - b_{cpt,x}\right) - 1\right] \cdot \xi_{cpt}\right] \cdot \chi_{sp} & \text{if } \frac{\left[\left(\rho_{pl,cpt} - \chi_{sp}\right) \cdot b_{cpt,x} \cdot \left(2 - b_{cpt,x}\right) - 1\right] \cdot \xi_{cpt}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{pl,cpt} - \chi_{sp}\right) \cdot b_{cpt,x} \cdot \left(2 - b_{cpt,x}\right) + \chi_{sp}\right] \cdot \alpha_{cpt}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.992$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sp,z} \cdot f_y}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sp,z} \cdot f_y}{\gamma_{M1}}}\right) + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.544$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,sps}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sps}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.511$$

EWM - Changed top plate stiffener thickness

This chapter covers the EWM when changing the top plate stiffener thickness to 14 mm

Geometry

Amount of:

Upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s,so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d,0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s,so}} = 350\,mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{so0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 14mm$ $s_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $s_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d \cdot b_d$ **parameters:** $A_s := 2 t_s \cdot h_s$ $A_w := 2 t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot s_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$E := 210GPa$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.119\,m^2$ -1 tab. 3.1

$A_c := A_w + A_d = 0.053\,m^2$ ue

$\varepsilon(\psi) := \sqrt{\frac{235MPa}{f_y(\psi)}}$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{\begin{aligned} &A_d \frac{t_d}{2} + A_{so}\left(t_d + \frac{h_{so}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ &+ A_b\left(t_d + h_s + h_{wy}\right) + A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{so0}\right) - \frac{h_{su.y}}{2}\right] \end{aligned}}{A_{tot}} = 369.102\,mm$$

Moment of inertia I_y

$s_{so0y} := \sin\left(21.12 \frac{\pi}{180}\right) s_{so0} = 7.206\,mm$

Stiffeners overlunder:

$I_{y.st.so} := n_{so} \frac{t_{so} h_{so}^3}{12} + n_{so} t_{so} h_{so}\left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.503 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2 t_{su} h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{so0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 8.864 \times 10^{-4}\,m^4$$
$$+ 2 n_{su} t_{su} h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{so0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2 n_{su} t_{su} h_{su}\left[t_d + h_s + \left(h_{wy} - s_{so0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 s_w h_w\left[h_{ts} - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 2.995 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d\left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.64 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b t_b^3}{12} + t_b b_b\left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.209 \times 10^{-7}\,m^4$

Side plate:

$I_{y.s} := 2 \frac{t_s h_s^3}{12} + 2 t_s h_s\left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.707 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} z - \sigma_{max}$

$\sigma_{1.neg}(\sigma_1, \sigma_2) := \max(\sigma_{neg}(\sigma_1), \sigma_{neg}(\sigma_2))$

$\sigma_{2.neg}(\sigma_1, \sigma_2) := \min(\sigma_{neg}(\sigma_1), \sigma_{neg}(\sigma_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} z + \sigma_{max}$

$\sigma_{1.pos}(\sigma_1, \sigma_2) := \max(\sigma_{pos}(\sigma_1), \sigma_{pos}(\sigma_2))$

$\sigma_{2.pos}(\sigma_1, \sigma_2) := \min(\sigma_{pos}(\sigma_1), \sigma_{pos}(\sigma_2))$

Buckling factor

$\psi_{neg}(\sigma_1, \sigma_2) := \frac{\sigma_{2.neg}(\sigma_1, \sigma_2)}{\sigma_{1.neg}(\sigma_1, \sigma_2)}$ $\psi_{pos}(\sigma_1, \sigma_2) := \frac{\sigma_{2.pos}(\sigma_1, \sigma_2)}{\sigma_{1.pos}(\sigma_1, \sigma_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi \equiv 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi \equiv 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi \equiv -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ 0.57 & \text{if } \psi \equiv 0 \\ 0.85 & \text{if } \psi \equiv -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$$k_{\sigma.4.2j}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi \equiv 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi \equiv -1 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(b_w, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(\psi) \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max(3 + \psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2) Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.011$

$$Condition := \begin{cases} 62\varepsilon(\psi_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \le -1 \\ \frac{42\varepsilon(\psi_s)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 7.731$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.505$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = -4.209 \times 10^{-3}\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 393.409\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 393.409\,mm$

$h_{s.eff.c1} := 0.4\, h_{s.eff.c} = 157.363\,mm$

$h_{s.eff.c2} := 0.6\, h_{s.eff.c} = 236.045\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\,mm$

$A_{s.eff.c} := 2\, h_{s.eff.c} t_s = 9.442 \times 10^3\,mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) t_s = 9.341 \times 10^3\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \frac{E \pi^2 t_s^2}{12\left(1 - \nu^2\right) b_s^2} = 1.395 \times 10^3\,MPa$ EC1993-1-5 eq. A.1

Stiffeners

$A_{sl} := t_d = 10\,mm$

$h_{sto} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.61$

$k_{\sigma,4.2b}(\psi_{so}) = 0.608$ Higher stress close to support

Class$_{sto}$:= "Class4" if $\frac{h_{so}}{t_{so}} > 21 \cdot \epsilon \cdot \left(k_{\sigma,4.2b}(\psi_{so})\right)$ = "Lower than class 4" "Lower than class 4" otherwise

$\lambda_{p,so} := \lambda_p\left(k_{\sigma,4.2b}(\psi_{so}), b_{so}, t_{so}\right) = 0.555$

$\rho_{d,so} := \rho_{4.4}\left(\lambda_{p,so}, \psi_{so}\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so,eff} := \rho_{d,so} \cdot b_{so} = 140\ mm$

$A_{so,eff} := n_{so} \cdot t_{so} \cdot b_{so,eff} = 1.764 \times 10^{-4}\ m^2$

Deck

$\frac{b_{sd}}{t_d} = 35 \quad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class$_{d}$:= "Class4" if $\frac{b_{sd}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4" "Lower than class 4" otherwise

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\ m$ $\psi_d := \psi_{neg}(z_d, z_d) = 1$

$b_{sd,so} := b_{sd} - t_{so} = 0.29\ m$ Tab. 5.2 ES1993-1-1] $k_{\sigma,d} := k_{\sigma,4.1}(\psi_d) = 4$

$\lambda_{p,d} := \lambda_p\left(k_{\sigma,d}, t_d, \epsilon_{cm}, t_d\right) = 0.727$

$\rho_d := \rho_{4.2}\left(\lambda_{p,d}, \psi_d\right) = 0.959$ Internal cross-sectional parts

$b_{so,eff} := \rho_d \cdot b_{c,so} = 322.298\ mm$

Areas:

$A_{sto} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(b_{so,eff} + t_d\right) = 5.04 \times 10^{-4}\ m^2$

$A_{c,eff,loc} := n_{so} \cdot t_{so,eff} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(b_{so,eff} + t_d\right) = 4.791 \times 10^{-4}\ m^2$

More than three longitudinal stiffeners --> $k_{\sigma,p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^{-5}\ m^4$

$y_{sp,sl} := \frac{n_{so} \cdot t_{so} \cdot b_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot b_{so} + b_d \cdot t_d} = 30.133\ mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp,sl}\right)^2\right] \dots + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp,sl}\right)^2\right] = 9.508 \times 10^{-7}\ m^4$

$\gamma := \frac{I_{sl}}{I_p} = 296.643 \quad \alpha := \frac{t}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot b_{so} \cdot t_{so} = 0.018\ m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035\ m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.504$

$k := \begin{cases} \frac{2\left[\left(1+\alpha^2\right)^2 + \gamma - 1\right]}{\left(1+\alpha^2\right)^2 \cdot (1+\delta) + \gamma - (1-\delta)} & \text{if } \alpha \le \sqrt{\gamma} \\ 4 & \text{otherwise} \end{cases} = 605.581$

$\chi_{op} := \begin{cases} 1 & \text{if } \overline{\lambda}_p \le 0.6 \cdot (3 + \psi) - 0.6 \cdot (1 - \psi) \cdot \delta \\ \frac{\left(3 \cdot \overline{\lambda}_p - \overline{\lambda}_p^2\right) \cdot (1+\delta)}{\sigma_{cr,p}} & \text{otherwise} \end{cases}$

$\rho = \rho_{4.2}\left(\overline{\lambda}_p, \psi\right) = 1 \cdot \delta$ Plate like behaviour

Critical buckling stress, orthotropic plate

Effective area of the stiffened plate (deck):

$A_{c,eff} := \rho_{4.2}\left(\overline{\lambda}_p, \psi\right) \cdot A_{c,eff,loc} + n_{so,eff} \cdot t_d = 5.113 \times 10^{-4}\ m^2$ EN1993-1-5 eq.A.1

$I_p = \frac{E \cdot t_d^3}{12 \cdot \left(1 - \nu^2\right)}$

Buckling strength

$N_{Rd} := A_{c,eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.815 \times 10^3\ kN$

Column like behaviour

$y_{sp,sl.c} := \frac{n_{so} \cdot b_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot b_{so} + n_{so} \cdot t_d} = 31.923\ mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp,sl.c}\right)^2\right] \dots + \left[\frac{t_{so} \cdot b_{so}^3}{12} + b_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp,sl.c}\right)^2\right] = 1.03 \times 10^{-7}\ m^4$

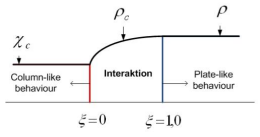
Buckling resistance stiffener

$A_{sto} = t_{so} \cdot b_{so} + n_{so} \cdot t_d = 5.46 \times 10^{-3}\ m^2$

$A_{sl,eff} := t_{so} \cdot b_{so,eff} + n_{so,eff} \cdot t_d = 5.183 \times 10^{-3}\ m^2$

$\beta_c := \frac{A_{sl,eff}}{A_{sl}} = 0.949$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr,sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot i^2} = 977.262\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sto})}{\sigma_{cr,sl}}} = 0.587$

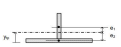
Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 131.96\ mm$

$e_c := \max\left[\left(y_{sp,sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{sp,sl}\right)\right] = 49.867\ mm$



$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e_c}} = 0.524$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot \left(\lambda_c - 0.2\right) + \lambda_c^2\right] = 0.774$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.783$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr,p}}{\sigma_{cr,sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.783$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{axrd} := n_{so} \cdot t_{so,eff} \cdot t_d \cdot \sigma_{neg}\left(t_d - \frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{b_{so,eff}}{2}\right) = -1.506 \times 10^4\ kN$

$\sigma_{com,Ed} := \frac{F_{axrd}}{n_{so} \cdot t_{so,eff} \cdot t_d + A_{so}} = -322.913\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sto})}{\gamma_M} = 277.8\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{sto})}{\gamma_M} > \sigma_{com,Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases}$ = "OK"

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d,eff} := n_{s,so} \cdot t_d \cdot t_{so,eff} = 0.032\ m^2$

$A_{tot,eff} := n_{so} \cdot t_{so,eff} + A_{d,eff} + A_w + A_{so} + A_b + A_{s,eff} = 0.116\ m^2$

$z_{tp,eff,neg} := \frac{\begin{aligned} &A_{d,eff} \cdot \frac{t_d}{2} + A_{so,eff} \cdot \left(t_d + \frac{b_{so,eff}}{2}\right) \dots \\ &+ 2 \cdot \left[t_s \cdot b_{s,eff,c1} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c1}}{2}\right) \dots \right. \\ &\left. + t_s \cdot b_{s,eff,c2} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c2}}{2}\right) + t_s \cdot b_{s,t} \cdot \left(t_d + b_s - \frac{b_{s,t}}{2}\right)\right] \dots \\ &+ A_w \cdot \left(t_d + b_s + \frac{b_{w,y}}{2}\right) + A_{so} \cdot \left[t_d + b_s + \frac{2}{3}\left(b_{w,y} - x_{so0}\right) - \frac{b_{so,y}}{2}\right] \dots \\ &+ A_b \cdot \left(t_{sb} - \frac{b_b}{2}\right) \end{aligned}}{A_{tot,eff}} = 377.797\ mm$

$I_{y,s,eff} := 2 \cdot \left[\frac{t_s \cdot b_{s,eff,c1}^3}{12} + t_s \cdot b_{s,eff,c1} \cdot \left(t_d + \frac{b_{s,eff,c1}}{2} - z_{tp,eff,neg}\right)^2 \dots + \frac{t_s \cdot b_{s,eff,c2}^3}{12} + t_s \cdot b_{s,eff,c2} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c2}}{2} - z_{tp,eff,neg}\right)^2 \dots + \frac{t_s \cdot b_{s,t}^3}{12} + t_s \cdot b_{s,t} \cdot \left(t_d + b_s - \frac{b_{s,t}}{2} - z_{tp,eff,neg}\right)^2\right] = 3.981 \times 10^{-4}\ m^4$

$I_{y,d,eff} := \frac{n_{s,so} \cdot t_{so,eff} \cdot t_d^3}{12} + n_{s,so} \cdot t_d \cdot t_{so,eff} \cdot \left(\frac{t_d}{2} - z_{tp,eff,neg}\right)^2 = 4.479 \times 10^{-3}\ m^4$

$I_{y,eff,neg} := I_{y,st,so} + I_{y,st,su} + I_{y,w} + I_{y,b} + I_{y,s} + I_{y,s,eff} + I_{y,d,eff} = 0.015\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{tot,eff,neg} := A_{tot,eff}$

$A_{c,eff,loc} = 4.791 \times 10^{-4}\ m^2$

$A_{c,eff,loc,mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{tot}}{\sigma_{com,Ed} \cdot \gamma_M} = -4.697 \times 10^{3}\ mm^2$

$A_{tot,eff,neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{eff,tot} := A_{c,eff} + A_{w,eff} + A_{so} + A_{so} + A_b = 1.069 \times 10^{-2}\ m^2$

Checkthe $\begin{cases} \text{"OK"} & \text{if } A_{c,eff,loc,mod} < A_{c,eff,loc} \\ \text{"EJ OK"} & \text{otherwise} \end{cases}$ = "OK" $= 407.118\ cm^2$

According to EC-EN 1993-1-5:4.5.1(7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

t_{so} Lower stiffener in tension -> no reduction

$t_{s,eff} := \begin{cases} \rho_c \cdot \frac{A_{c,eff,loc,mod}}{A_{c,eff,loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.825\ mm$

$y_{sp,eff,neg} := \frac{\begin{aligned} &n_{so} \cdot b_{so,eff} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so,eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot n_{so,eff} \cdot t_{d,eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot b_{s,eff,c1} \cdot \left(t_d + \frac{b_{s,eff,c1}}{2}\right) \dots \\ &+ t_s \cdot b_{s,eff,c2} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c2}}{2}\right) + t_s \cdot b_{s,t} \cdot \left(t_d + b_s - \frac{b_{s,t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + b_s + \frac{b_{w,y}}{2}\right) + A_{so} \cdot \left[t_d + b_s + \frac{2}{3}\left(b_{w,y} - x_{so0}\right) - \frac{b_{so,y}}{2}\right] \dots \\ &+ A_b \cdot \left(t_{sb} - \frac{b_b}{2}\right) \end{aligned}}{A_{eff,tot}} = 398.066\ mm$

Effective moment of inertia:

$I_{y,d,eff,fin} := \frac{n_{s,so} \cdot t_{so,eff} \cdot t_d^3}{12} + t_d \cdot n_{s,so} \cdot t_{so,eff} \cdot \left(\frac{t_d}{2} - z_{tp,eff,neg}\right)^2 = 4.98 \times 10^{-3}\ m^4$

$z_{web,fin} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot t_{so0} = 7.206\ mm$

Stiffeners overlunder:

$I_{y,st,su,fin} := n_{so} \cdot \frac{t_{so} \cdot b_{so,eff}^3}{12} + n_{so} \cdot t_{so} \cdot b_{so,eff} \cdot \left(t_d + \frac{b_{so,eff}}{2} - z_{tp,eff,neg}\right)^2 = 1.813 \times 10^{-3}\ m^4$

$I_{y,st,su,1.fin} = 6.1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y,st,su,fin} := I_{y,st,su,1.fin} \dots + 2 \cdot t_{so} \cdot b_{so} \cdot \left[\left(t_d + b_s + \frac{1}{3}\left(b_{w,y} - x_{so0y}\right) - \frac{b_{so,y}}{2} - z_{tp,eff,neg}\right)^2 \dots + \left(t_d + b_s + \frac{2}{3}\left(b_{w,y} - x_{so0y}\right) - \frac{b_{so,y}}{2} - z_{tp,eff,neg}\right)^2 \dots + \left(t_d + b_s + \left(b_{w,y} - x_{so0y}\right) - \frac{b_{so,y}}{2} - z_{tp,eff,neg}\right)^2\right] = 7.479 \times 10^{-4}\ m^4$

Web:

$I_{y,w,1.fin} = 2 \cdot 0.00031 m^4$ from FEM

$I_{y,w,fin} = I_{y,w,1.fin} + 2 \cdot t_w \cdot b_w \cdot \left[t_{sb} - \frac{t_b}{2} - \frac{1}{2}\left(b_{w,y}\right) - z_{tp,eff,neg}\right]^2 = 2.516 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y,b,fin} = \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(t_{sb} - z_{tp,eff,neg} - \frac{t_b}{2}\right)^2 = 3.747 \times 10^{-3}\ m^4$

Side plate:

$I_{y,s,fin} := 2 \cdot \left[\frac{t_s \cdot b_{s,eff,c1}^3}{12} + t_s \cdot b_{s,eff,c1} \cdot \left(t_d + \frac{b_{s,eff,c1}}{2} - z_{tp,eff,neg}\right)^2 \dots + \frac{t_s \cdot b_{s,eff,c2}^3}{12} + t_s \cdot b_{s,eff,c2} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c2}}{2} - z_{tp,eff,neg}\right)^2 \dots + \frac{t_s \cdot b_{s,t}^3}{12} + t_s \cdot b_{s,t} \cdot \left(t_d + b_s - \frac{b_{s,t}}{2} - z_{tp,eff,neg}\right)^2\right] = 4.675 \times 10^{-4}\ m^4$

$I_{y,eff,neg,fin} := I_{y,d,eff,fin} + I_{y,st,su,fin} + I_{y,w,fin} + I_{y,b,fin} + I_{y,s,fin} = 1.427 \times 10^{10}\ mm^4$

Load

$q_{-}M := 89 \frac{kN}{m^2}$

$Q_{-}M := q_{-}M \cdot b_d = 311.5 \frac{kN}{m}$ $V_{Ed} := \frac{Q_{-}M \cdot l_{bridge}}{2} = 2.804 \times 10^3\ kN$

$M_{Ed} := \frac{Q_{-}M \cdot l_{bridge}^2}{8} = 1.262 \times 10^4\ kN \cdot m$

Moment capacity

$W_{y,eff} := \frac{I_{y,eff,neg,fin}}{z_{tp,eff,neg}} = 0.036\ m^3$

$M_{Rd} := W_{y,eff} \cdot \frac{f_y(t_d)}{\gamma M1} = 1.273 \times 10^4\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.991$

Shear capacity

$\eta_1 := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot t\left(t_{wb}\right) + t_w = 0.293\,m$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wg}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{wdl} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{wa}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{sa}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{sa} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{sa} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 \cdot t_{sa} \cdot h_{sa}^3 \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15ct each side of the stiffeners:

$\frac{b_b}{2} + s_{wdl} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{wa}}{2} = 0.525\,m$

$a_{st.z.1} := a_{st.z.1} + s_{sa} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$

$a_{st.z.2} := a_{st.z.2} + s_{sa} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$

Moment of inirtia of 15ct of the web z-direction

$I_{stw.z} := 2 h_{web} \cdot t_{wa}^3 \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{wi} = 0$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4 \cdot t_w \cdot \varepsilon\left(t_w\right) \cdot \sqrt{k_{\tau 1}}} = 0.455$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[\eta \cdot \left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_w, t_{wb}\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_w, t_{wb}\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$$k_\tau := \begin{cases} 5.34 + 4 \cdot \left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_w, t_{wb}\right)^3 \cdot h_{web}} + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_w, t_{wb}\right)^3 \cdot h_{web}}} & \text{otherwise} \\ \alpha^2 \end{cases} = 1.635 \times 10^3$$

$\lambda_w := \max\left(\frac{h_w}{37.4 \cdot t_w \cdot \varepsilon\left(t_w\right) \cdot \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$, has to be performed for webs and webs SS-EN 1993-1-5 5.3 (5.6)

$\gamma_{M1} := 1.2$ EC1993-1-3-5 ANM.2 5.1

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ SS-EN 1993-1-5 5.3 Table 5.1, $\frac{i}{t_w} \leq 31 \cdot \frac{\varepsilon\left(t_w\right)}{\eta} \cdot \sqrt{k_{\tau 1}}$ stiff end stiffener = "Reduce"

EC1003-1-5 Ch. 5.1(2).

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

S83 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y\left(t_w\right) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.804 \times 10^3\,kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.66$ Contribution from two web

$V_{b.Rd} \leq \frac{\eta \cdot f_y\left(t_w\right) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{pl.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 393.066\,mm$

$a_{sl.eff.neg} := h_h - \frac{t_h}{2} - z_{tp.eff.neg} = 483.934\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{pl.eff.neg} \cdot \left|a_{pl.eff.neg}\right| \cdot f_y\left(t_d\right)}{\gamma_{M0}} + \frac{A_{sl.eff.neg} \cdot \left|a_{sl.eff.neg}\right| \cdot f_y\left(t_h\right)}{\gamma_{M0}} = 7.246 \times 10^3\,kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{pl} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{t_v \cdot \left(h_v + h_{wg}\right)^2 \cdot f_y\left(t_v\right)}\right] = 0.623\,m$

$c_{sl} := a \cdot \left[0.25 + \frac{1.6 \cdot b_h \cdot t_h^2 \cdot f_y\left(t_h\right)}{t_v \cdot \left(h_v + h_{wg}\right)^2 \cdot f_y\left(t_v\right)}\right] = 0.59\,m$

$V_{bf.Rd.creed} := \frac{b_d \cdot t_d^2 \cdot f_y\left(t_d\right)}{c_{pl} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_h \cdot t_h^2 \cdot f_y\left(t_h\right)}{c_{sl} \cdot \gamma_{M1} \cdot \gamma_n} = 294.676\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.creed} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\,kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y\left(t_w\right) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{Rd} = V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3\,kN$

Check:

$\frac{V_{Ed}}{2} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.66$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.tot} := I_{y.d.eff.tot} + I_{y.h.tot} = 8.727 \times 10^{-3}\,m^4$

The flanges effective area:

$A_{eff.f.tot} := n_{so.eff} \cdot t_d + n_{wa} \cdot s_{so.eff} \cdot t_{d.eff} + n_{wa} \cdot t_{wa} \cdot t_{d.eff} + A_h = 429.079\,cm^2$

$z_{tp.f.eff.neg} := \frac{n_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + \ldots}{A_{eff.f.tot}} = 331.426\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.tot}}{z_{tp.f.eff.neg}} = 0.026\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 9.348 \times 10^3\,kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.273 \times 10^4\,kN \cdot m$

Moment capacity check when $q_{_}M = 89\,\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.991$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The reduced stress method

-Changed top plate stiffener thickness to 14 mm

Material

$f_{y0} := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ 335\text{MPa} & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} = 1$

$E = 210\text{GPa}$

$\nu = 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\,\text{GPa}$

Geometry

Bottom plate:

$t_b = 16\text{mm}$ $b_b = 1000\text{mm}$ $A_b = b_b t_b = 0.016\text{m}^2$

Top plate:

$t_t = 10\text{mm}$ $b_t = 3500\text{mm}$ $A_t = b_t t_t = 0.035\text{m}^2$

Side plates:

$t_s = 12\text{mm}$ $h_s = 389.2\text{mm}$ $A_s = h_s t_s = 4.67\cdot 10^{-3}\,\text{m}^2$

Webs:

$t_w = 12\text{mm}$ $h_w = 1340\text{mm}$ $A_w = h_w t_w = 0.016\text{m}^2$

$h_{w,k} = 1258\text{mm}$ $h_{w,fl} = 482.8\text{mm}$

Top stiffeners:

$t_{st} = 14\text{mm}$ $b_{st} = 140\text{mm}$ $A_{st} = h_{st} t_{st} = 1.96\cdot 10^{-3}\,\text{m}^2$

$s_{st} = 350\text{mm}$ $n_{st} = 9$

Web stiffeners:

$t_{st} = 12\text{mm}$ $h_{st} = 120\text{mm}$ $A_{st} = h_{st} t_{st} = 1.44\cdot 10^{-3}\,\text{m}^2$

$s_{st} = 440\text{mm}$ $n_{st} = 3$

Whole bridge:

$t_{bridge} = 18\text{m}$ $h_b = \frac{h_s}{2} + h_{w,fl} + h_t + t_b = 0.89\text{m}$ $n_{br} = 2\text{m}$

$I_{yy} = 0.0149\text{m}^4$ From grasshopper

$V_{fg} = 0.526956\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q = 74.4\,\frac{\text{kN}}{\text{m}}$

$Q := q\cdot t_{br} = 259\,\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot t_{bridge}^2}{8} = 1.049\cdot 10^4\,\text{kN m}$

$V_{Ed} := \frac{Q\cdot t_{bridge}}{2} = 2.331\cdot 10^3\,\text{kN}$

Top plate:

$\sigma_{x,Ed,up} := \frac{M_{Ed}}{I_{yy}}\left(h_w - \frac{t_t}{2} - y_g\right) = 261.73\,\text{MPa}$ $\sigma_{z,Ed,up} = 0$

$S_{yy} := \frac{I_y t_t}{h_w - 2}\left(h_w - \frac{t_t}{2} - y_g\right) = 1.158\cdot 10^{-3}\,\text{m}^3$

$\tau_{Ed,up} := \frac{V_{Ed} S_{yy}}{I_{yy} 4\text{m}\left(t_t + t_{st}\right)} = 15.422\,\text{MPa}$

Side plates:

$\sigma_{x,Ed,up} := \frac{M_{Ed}}{I_{yy}}\left(h_w - t_t - y_g\right) = 258.135\,\text{MPa}$

$\sigma_{z,Ed,up} := \frac{Q}{A_s} = 10.792\,\text{MPa}$

$S_{yy} := \left(h_w - t_t - y_g\right)\cdot t_s\cdot\frac{\left(h_w - t_t - y_g\right)}{2} + \frac{t_t t_s}{2}\left(h_w - \frac{t_t}{2} - y_g\right) = 9.694\cdot 10^{-3}\,\text{m}^3$
$+ \frac{h_{st}}{s_{st}} t_{st} t_s\left(h_w - t_t - \frac{h_{st}}{2} - y_g\right)$

$S_{yy} := \frac{\left(h_w - \frac{t_w t_{w,k}}{2} - 2\cdot\frac{t_s t_{st}}{2}\right)}{n_{br}\cdot 2}\left(h_w - t_t - \frac{h_{st}}{4} - y_g\right) = 1.468\cdot 10^{-3}\,\text{m}^3$
$+ \frac{t_t t_s}{n_{br}\cdot 2}\left(h_w - \frac{t_t}{2} - y_g\right)$

Twn webs stiffeners:

$\tau_{Ed,up} := \frac{V_{Ed} S_{yy}}{I_{yy} 2\cdot t_w} = 16.749\,\text{MPa}$

$\sigma_{Ed,up} := \sqrt{\sigma_{x,Ed,up}^2 + \sigma_{z,Ed,up}^2 - \sigma_{x,Ed,up}\sigma_{z,Ed,up} + 3\tau_{Ed,up}^2} = 258.135\,\text{MPa}$

Governing Von Mises from FE-software

$\sigma_{V,LNDT} := 4.56\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_{cr} := \sigma_{V,LNDT} q = 337.44\,\text{MPa}$

$\alpha_{ult} := \frac{f_y[1]}{\sigma_{cr}} = 1.052$

$\alpha_{cr,LNDT} := 295.149\,\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr,LNDT}}{q} = 3.972$

$\lambda_p := \sqrt{\frac{f_{y0}}{\sigma_{cr}}} = 0.616$ Slenderness for all parts

Top plate check

$\Phi_{tp} = 1$

$\rho_{p,tp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\Phi_{tp})}{\lambda_p^2} & \text{if } \lambda_p \ge 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E,tp} := \frac{\pi^2 E t_t^2}{12\cdot\left(1-\nu^2\right)\cdot b_t^2} = 1.549\,\text{MPa}$

$I_{y,tp} := \frac{b_t t_t^3}{12\cdot\left(1-\nu^2\right)} = 3.205\cdot 10^{-7}\,\text{m}^4$

$y_{g,tp} := \frac{A_t\frac{t_t}{2} + n_{st} A_{st}\left(t_t + \frac{h_{st}}{2}\right)}{A_t + n_{st} A_{st}} = 30.133\,\text{mm}$

$I_{z,tp} := \frac{b_t t_t^3}{12} + A_t\left(y_{g,tp} - \frac{t_t}{2}\right)^2 + \ldots = 9.508\cdot 10^{-7}\,\text{mm}^4$
$+ n_{st}\left[\frac{t_{st} h_{st}^3}{12} + A_{st}\left(t_t + \frac{h_{st}}{2} - y_{g,tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{z,tp}}{I_{y,tp}} = 296.643$

$k_{\sigma} := \frac{n_{st} A_{st}}{A_t} = 0.504$

$\alpha_{tp} := \min\left(\frac{2\pi}{h_w}, 0.5\right) = 0.571$

$k_{\sigma,p,tp} := \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\left(\gamma_{tp}-1\right)\left(1+k_{\sigma}\right)} & \text{if } \alpha_{tp} < \sqrt{\gamma_{tp}} \\ \frac{4\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\gamma_{tp}+1\right)\left(1+k_{\sigma}\right)} & \text{if } \alpha_{tp} \ge \sqrt{\gamma_{tp}} \end{cases} = 605.581$

$\sigma_{cr,p,tp} := k_{\sigma,p,tp}\sigma_{E,tp} = 938.281\,\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1,eff,tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}} c_{tp} = 0.175\text{m}$

$b_{2,eff,tp} := \frac{2}{5-\psi_{tp}} c_{tp} = 0.175\text{m}$ Parts of the top plate that contributes to the stiffener

$b_{c,t,tp} := b_{1,eff,tp} + b_{2,eff,tp} + t_{st} = 0.35\text{m}$

$y_{g,c,t,tp} := \frac{t_t b_{c,t,tp}\frac{t_t}{2} + A_{st}\left(t_t + \frac{h_{st}}{2}\right)}{t_t b_{c,t,tp} + A_{st}} = 0.032\text{m}$ combined center of gravity for stiffener and contributing top plate.

$I_{c,t,tp} := \frac{b_{c,t,tp} t_t^3}{12} + b_{c,t,tp} t_t\left(t_t - y_{g,c,t,tp} - \frac{t_t}{2}\right)^2 + \frac{t_{st} h_{st}^3}{12} + A_{st}\left(t_t + \frac{h_{st}}{2} - y_{g,c,t,tp}\right)^2$

Combined moment of inertia

$\sigma_{cr,c,tp} := \frac{\pi^2 E i_{c,t,tp}}{A_{c,t,tp}\cdot s_{st}^2} = 977.262\,\text{MPa}$ Critical stress for kolummlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,p} = \sigma_{cr,c}$

$\sigma_{cr,c,tp} := \sigma_{cr,c,tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{c,t,tp}}{A_{c,t,tp}}} = 0.043\text{m}$

$e_{tp} := \max\left(y_{g,c,t,tp} - y_g + \frac{h_{st}}{2} - y_{g,c,t,tp}\right) = 0.048\text{m}$

$\alpha_{e,tp} := 0.49 + \frac{0.09}{\frac{2e_{tp}}{h_{st}}} = 0.59$ stiffener with open cross section (curve c)

$\Phi_{tp} := 0.5\left[1 + \alpha_{e,tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.812$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\Phi_{tp} + \sqrt{\Phi_{tp}^2 - \lambda_p^2}}, 1\right) = 0.745$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr,tp,up}}{\sigma_{cr,c,tp}} - 1.0, 0\right), 1\right) = 0$

$\rho_{c,tp} := \begin{cases} \left[\left(\rho_{p,tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.745$

Shear

$\eta_t = 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta_t & \text{if } \lambda_w < \frac{0.83}{\eta_t} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta_t} \le \lambda_w < 1.08 \\ \frac{1.37}{\left(0.7 + \lambda_w\right)} & \text{if } \lambda_w \ge 1.08 \end{cases} = 1.2$

$\frac{\rho_{c,tp}\alpha_{ult}}{\gamma_{M1}} = 0.784$

Side plates check

x-direction

$\sigma_{max} := f_y[t_s]$

$\kappa_{\sigma 0} := \left|\frac{-\sigma_{min}}{f_y[t_s]} + \sigma_{max}\right| - 1$

$\psi_{tp,s} := \frac{\sigma_t\left(h_w - t_t - h_t\right)}{\sigma\left(h_w - t_t\right)} = -0.084$

$\sigma_{x,top} := \frac{M_{Ed}}{I_{yy}}\left(h_w - t_t - y_g\right) = 258.135\,\text{MPa}$

$\sigma_{x,bot} := \frac{M_{Ed}}{I_{yy}}\left(h_w - t_t - h_t - y_g\right) = -21.681\,\text{MPa}$

$\psi_{sp1,x} := \max\left(3 - \psi_{tp,x}, 0\right) = 2.916$ $\psi_{sp1} := \frac{\sigma_{x,bot}}{\sigma_{x,top}} = -0.084$

$\rho_{p,sp,x} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(\psi_{sp1,x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

$k_{\sigma,sp,x} := \begin{cases} 1 & \text{if } \psi_{tp,x} \ge 1 \\ \frac{8.2}{\left(1.05 + \psi_{tp,x}\right)} & \text{if } 1 > \psi_{tp,x} > 0 \\ 7.81 & \text{if } \psi_{tp,x} = 0 \\ 7.81 - 6.29\,\psi_{tp,x} + 9.78\,\psi_{tp,x}^2 & \text{if } 0 > \psi_{tp,x} > -1 \\ 23.9 & \text{if } \psi_{tp,x} = -1 \\ 5.98\left(1 - \psi_{tp,x}\right)^2 & \text{if } -1 > \psi_{tp,x} > -3 \\ 95.68 & \text{if } \psi_{tp,x} < -3 \end{cases} = 8.407$

$\sigma_{cr,p,sp,x} := \sigma_{E,sp,x} k_{\sigma,sp,x} = 1.517\cdot 10^3\,\text{MPa}$

Column-like behaviour

$\sigma_{cr,c,sp,x} := \frac{\pi^2 E i_s^2}{12\cdot\left(1-\nu^2\right)\cdot s_{st}^2} = 6.833\,\text{MPa}$

$\alpha_{sp} = 0.21$

$\Phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.733$

$\chi_{sp} := \min\left(\frac{1}{\left(\Phi_{sp} + \sqrt{\Phi_{sp}^2 - \lambda_p^2}\right)}, 1\right) = 0.884$

Interpolation

$\xi_{sp,x} := \min\left(\max\left(\frac{\sigma_{cr,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0, 0\right), 1\right) = 1$

$\rho_{c,sp,x} := \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2-\xi_{sp,x}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2-\xi_{sp,x}\right) + \chi_{sp}\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2-\xi_{sp,x}\right) + \chi_{sp}\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

Z-direction

$\psi_{tp,z} = 1$

$\alpha_{p,sp} = 0.34$ $\lambda_{p0,sp} = 0.7$

$\Phi_{p,sp} := 0.5\left[1 + \alpha_{p,sp}\left(\lambda_p - \lambda_{p0,sp}\right) + \lambda_p^2\right] = 0.794$

$\rho_{p,sp,z} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\Phi_{p,sp} + \sqrt{\Phi_{p,sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,p,sp,z} = 4$

$\sigma_{E,sp,z} := \frac{\pi^2 E t_s^2}{12\cdot\left(1-\nu^2\right)\cdot s_{st}^2} = 6.833\,\text{MPa}$

$\sigma_{cr,p,sp,z} := k_{\sigma,p,sp,z}\sigma_{E,sp,z} = 27.331\,\text{MPa}$

Column-like behaviour

$\sigma_{cr,c,sp,z} := \frac{\pi^2 E i_s^2}{12\cdot\left(1-\nu^2\right)\cdot s_{st}^2} = 180.432\,\text{MPa}$

Interpolation

$\xi_{sp,z} := \min\left(\max\left(\frac{\sigma_{cr,sp,tp,z}}{\sigma_{cr,c,sp,z}} - 1.0, 0\right), 1\right) = 0$

$\rho_{c,sp,z} := \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2-\xi_{sp,z}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2-\xi_{sp,z}\right) + \chi_{sp}\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2-\xi_{sp,z}\right) + \chi_{sp}\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.884$

Top plate stiffeners - check

x-direction

$\rho_{p,t,sp} := \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$\psi_{sp,t,x} := \frac{\sigma\left(h_w - t_t - h_{st}\right)}{\sigma\left(h_w - t_t\right)} = 0.61$

$$\sigma_{E,cp,x} := \frac{\pi^2 \cdot E_{cm}^{\ 2}}{12 \cdot \left(1 - \nu^2\right) \cdot \lambda_{bx}^{\ 2}} = 1.898 \times 10^3 \ \text{MPa}$$

$$k_{\sigma,cp,x} := \begin{cases} 1 & \text{if } \psi_{cp,x} \neq 1 \\ \dfrac{8.2}{1.05 + \psi_{cp,x}} & \text{if } 1 > \psi_{cp,x} > 0 \\ 7.81 & \text{if } \psi_{cp,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{cp,x} + 9.78 \cdot \psi_{cp,x}^{\ 2} & \text{if } 0 > \psi_{cp,x} > -1 \\ 23.9 & \text{if } \psi_{cp,x} = -1 \\ 5.98 \cdot \left(1 - \psi_{cp,x}\right)^2 & \text{if } -1 > \psi_{cp,x} > -3 \\ 95.68 & \text{if } \psi_{cp,x} < -3 \end{cases} = 4.042$$

$$\sigma_{cr,cp,cp,x} := k_{\sigma,cp,x} \cdot \sigma_{E,cp,x} = 7.671 \times 10^3 \ \text{MPa}$$

Column-like behaviour

$$\sigma_{cr,c,cp,x} := \frac{\pi^2 \cdot E_{cm}^{\ 2}}{12 \cdot \left(1 - \nu^2\right) \cdot \lambda_{bx}^{\ 2}} = 9.3 \ \text{MPa}$$

$$\chi_{cp,x} := \chi_p$$

Interpolation

$$\xi_{cp,x} := \min\left(\max\left(\frac{\sigma_{cr,cp,cp,x}}{\sigma_{cr,c,cp,x}} - 1.0, 0\right), 1\right) = 1$$

$$\chi_{c,cp,x} := \begin{cases} \left[\left(\chi_{bx,cp} - \chi_{pb}\right) \cdot k_{cp,x} \cdot \left(2 - k_{cp,x}\right) + \chi_{pb}\right] & \text{if } \frac{\left[\left(\chi_{bx,cp} - \chi_{pb}\right) \cdot k_{cp,x} \cdot \left(2 - k_{cp,x}\right) + \chi_{pb}\right] \cdot \gamma_{M0}}{\gamma_{M1}} \cdot 1 < 1 \\ 1 & \text{if } \frac{\left[\left(\chi_{bx,cp} - \chi_{pb}\right) \cdot k_{cp,x} \cdot \left(2 - k_{cp,x}\right) + \chi_{pb}\right] \cdot \gamma_{M0}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right)^2 = 0.983$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{x,Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right) = 0.574$$
$$+ 3 \cdot \left(\frac{\tau_{Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,cp}}{\frac{f_{yk,cp,x}}{\gamma_{M1}}}\right)^2 = 0.533$$

EWM - Changed top plate stiffener thickness

This chapter covers the EWM when changing the top plate stiffener thickness to 18 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s,so} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d,0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s,so}} = 350\,mm$

Side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140mm$ $t_{su} := 18mm$ $s_{su} := 6mm$

Stiffeners under: $h_{sl} := 120mm$ $t_{sl} := 12mm$ $s_{sl} := 5mm$

$h_{sl,y} := h_{sl} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{tt} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d \cdot b_d$ **Parameter:** $A_s := 2 t_s \cdot h_s$ $A_w := 2 t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$ $A_{sl} := n_{sl} \cdot t_{sl} \cdot h_{sl}$

$E := 210GPa$

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.124\,m^2$ tab. 3.1

$A_C := A_{tot} + A_d = 0.058\,m^2$ **A**

$\alpha(t) := \sqrt{\frac{235MPa}{f_y(t)}}$ **αM**

Change of center of gravity, z_{sp}

$$z_{sp} := \frac{\begin{pmatrix} A_d \frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ + A_b\left(t_d + h_s + h_{wy}\right) + A_{sl}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{sl,y}}{2}\right] \end{pmatrix}}{A_{tot}} = 357.335\,mm$$

Moment of inertia I_y

$s_{su0,y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206\,mm$

Stiffeners over/under:

$I_{y,st.so} := n_{su} \cdot \frac{t_{su} \cdot h_{su}^3}{12} + n_{su} \cdot t_{su} \cdot h_{su}\left(t_d + \frac{h_{su}}{2} - z_{sp}\right)^2 = 1.781 \times 10^{-3}\,m^4$

$I_{y,st.su.1} := 6 \cdot 1.586 \cdot 10^{-6}\,m^4$ from FEM

$I_{y,st.su} := I_{y,st.su.1} + 2 t_{su} h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \dots = 9.468 \times 10^{-4}\,m^4$
$+ 2 t_{su} h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \dots$
$+ 2 t_{su} h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.00312\,m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 t_w h_w\left[h_s - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{sp}\right]^2 = 3.204 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d\left(\frac{t_d}{2} - z_{sp}\right)^2 = 4.345 \times 10^{-7}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b t_b^3}{12} + t_b b_b\left(h_{tt} - z_{sp} - \frac{t_b}{2}\right)^2 = 4.405 \times 10^{-7}\,m^4$

Side plate:

$I_{y.s} := 2 \frac{t_s h_s^3}{12} + 2 t_s h_s\left(t_d + \frac{h_s}{2} - z_{sp}\right)^2 = 3.358 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{sp}} z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{sp}} z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi \equiv 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi \equiv 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi \equiv -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$$k_{\sigma.4.2u}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ 0.57 & \text{if } \psi \equiv 0 \\ 0.85 & \text{if } \psi \equiv -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$$k_{\sigma.4.2l}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi \equiv 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi \equiv -1 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(b_p, h, t) := \frac{\frac{b_p}{h}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max(3 + \psi), 0}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2) Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.121$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.71$

$s_{s.so} := s_{so} - s_{so}$ Table 5.2 ES1993-1-1

$\lambda_{p.s} := \lambda_p\left(h_{\sigma.4.1}(\psi_s), h_s, t_s\right) = 0.476$

$\rho_s := \rho_{4.2}\left(\lambda_{p.s}, \psi_s\right) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{sp} = 0.042\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 347.335\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s\left(h_{s.c}\right) = 347.335\,mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 138.934\,mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 208.401\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\,mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.336 \times 10^{-3}\,mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right) \cdot t_s = 9.341 \times 10^{-3}\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_d)\cdot\frac{E\cdot\pi^2\cdot t_d^{\,2}}{12\cdot(1-\nu^2)\cdot b_d^{\,2}} = 1.572\times10^{3}\cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$t_{so} := t_d = 10\cdot mm$

$b_{so} := t_d + b_{so} = 150\cdot mm$

$\psi_{so} := \frac{\sigma_{sup}(z_s)}{\sigma_{sup}(z_1)} = 0.597$

$k_{\sigma.4.2}(\psi_{so}) = 0.617$ Higher stress close to support

$Class4_{so} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{so}}{t_{so}} > 21\cdot\varepsilon\cdot\sqrt{k_{\sigma.4.2}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2}(\psi_{so}), b_{so}, t_{so}) = 0.429$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so}\cdot b_{so} = 140\cdot mm$

$A_{so.eff} := n_{so}\cdot t_{so}\cdot b_{so.eff} = 2.268\times10^{4}\cdot mm^2$

Deck

$\frac{b_{sd}}{t_d} = 35 \qquad 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{sd} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{sd}}{t_d} > 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5\times10^{-3}\cdot m \qquad \psi_d := \psi_{sup}(z_d, z_d) = 1$

$n_{sd} := n_{so} = 1$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{x.so}, t_d) = 0.718$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.966$ Internal cross-sectional parts

$s_{so.eff} := \rho_d\cdot s_{x.so} = 320.613\cdot mm$

Areas:

$A_{sd} := n_{sd}\cdot s_{so}\cdot t_d + n_{so}\cdot t_{so}\cdot(b_{so.eff} + t_d) = 5.58\times10^{-4}\cdot m^2$

$A_{c.eff.loc} := n_{sd}\cdot s_{so.eff}\cdot t_d + n_{so}\cdot t_{so}\cdot(b_{so.eff} + t_d) = 5.316\times10^{-4}\cdot m^2$

More than three longitudinal stiffeners -> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d\cdot t_d^{\,3}}{12\cdot(1-\nu^2)} = 3.205\times10^{-5}\cdot m^4$

$y_{sp.sl} := \frac{n_{sd}\cdot t_{so}\cdot b_{so}\cdot\left(t_d + \frac{b_{so}}{2}\right) + b_d\cdot t_d\cdot\frac{t_d}{2}}{n_{sd}\cdot t_{so}\cdot b_{so} + b_d\cdot t_d} = 34.49\cdot mm$

$I_{sd} := \left[\frac{b_d\cdot t_d^{\,3}}{12} + b_d\cdot t_d\cdot\left(\frac{t_d}{2} - y_{sp.sl}\right)^2\right] \cdots = 1.147\times10^{-6}\cdot m^4$
$+ \left[\frac{n_{sd}\cdot t_{so}\cdot b_{so}^{\,3}}{12} + n_{so}\cdot b_{so}\cdot t_{so}\cdot\left(t_d + \frac{b_{so}}{2} - y_{sp.sl}\right)^2\right]$

$\gamma := \frac{I_{sd}}{I_p} = 358.013 \qquad \alpha := \frac{a}{b_d} = 0.571$

$k_{\sigma.p} := \begin{cases} \frac{2\cdot\left[(1+\alpha^2)^2 + \gamma - 1\right]}{\alpha^2\cdot(\psi+1)\cdot(1+\delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4\cdot(1-\sqrt{\alpha})}{(\psi+1)\cdot(1+\delta)} & \text{otherwise} \end{cases} = 666.712$ 2)

$A_p := b_d\cdot t_d = 0.035\cdot m^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p}\cdot\frac{E\cdot\pi^2\cdot t_d^{\,2}}{12\cdot(1-\nu^2)\cdot b_d^{\,2}} = 1.033\times10^{3}\cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.953$

$\lambda_p := \sqrt{\frac{\beta_{A.c}\cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.572$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi)\cdot A_{c.eff.loc} + n_{sd.eff}\cdot t_d = 5.636\times10^{-4}\cdot m^2$

Bucking strength

$N_{Rd} := A_{c.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 2.001\times10^{4}\cdot kN$

Column like behaviour

$y_{sp.s.c} := \frac{t_{so}\cdot b_{so}\cdot\left(t_d + \frac{b_{so}}{2}\right) + n_{so}\cdot t_d\cdot\frac{t_d}{2}}{t_{so}\cdot b_{so} + n_{so}\cdot t_d} = 36.395\cdot mm$

$I_{sd.c} := \left[\frac{s_{so}\cdot t_d^{\,3}}{12} + s_{so}\cdot t_d\cdot\left(\frac{t_d}{2} - y_{sp.s.c}\right)^2\right] \cdots = 1.239\times10^{-7}\cdot m^4$
$+ \left[\frac{t_{so}\cdot s_{so}^{\,3}}{12} + t_{so}\cdot s_{so}\cdot\left(t_d + \frac{s_{so}}{2} - y_{sp.s.c}\right)^2\right]$

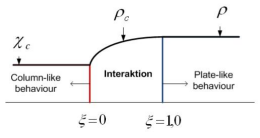
Bucking resistance stiffener

$A_{s.sl} := t_{so}\cdot b_{so} + s_{so}\cdot t_d = 6.02\times10^{-3}\cdot mm^2$

$A_{s.sl.eff} := t_{so}\cdot b_{so.eff} + s_{so.eff}\cdot t_d = 5.726\times10^{-3}\cdot mm^2$

$\beta_s := \frac{A_{s.sl.eff}}{A_{sl}} = 0.951$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2\cdot E\cdot I_{sl.c}}{A_{sl}\cdot s^2} = 1.066\times10^{3}\cdot MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_s\cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.563$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 138.062\cdot mm$

$\bar{e}_c := \max\left[\left(y_{sp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{b_{so}}{2} - y_{sp.sl}\right)\right] = 45.51\cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.52$

$\phi := 0.5\cdot\left[1 + \alpha_e\cdot(\lambda_c - 0.2) + \lambda_c^{\,2}\right] = 0.753$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^{\,2}}}\right) = 0.799$

Interaction

$\xi := \max\left(\frac{\sigma_{cr.pl}}{\sigma_{cr.sl}} - 1, 1\right), 0) = 0$

ξ is zero --> column like behvieur

Interaction between the two types of behaviour (column/plate bucking)

$\rho_c := (\rho - \chi_c)\cdot\xi\cdot(2 - \xi) + \chi_c = 0.799$

Checks

Mean stress ratt

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{axsl} := n_{so}\cdot s_{so.eff}\cdot t_d\cdot\sigma_{sup}\left(\frac{t_d}{2}\right) + A_{so}\cdot\sigma_{sup}\left(t_d + \frac{b_{so.eff}}{2}\right) = -1.635\times10^{4}\cdot kN$

$\sigma_{com.Ed} := \frac{F_{axsl}}{n_{so}\cdot s_{so.eff}\cdot t_d + A_{so}} = -317.242\cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c\cdot f_y(t_{so})}{\gamma_M} = 283.468\cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c\cdot f_y(t_{so})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{sd}\cdot s_{so.eff} = 0.032\cdot m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{s.eff} = 0.121\cdot m^2$

$z_{sp.eff.neg} := \frac{\begin{aligned} &A_{sl.eff}\cdot\left(\frac{t_d}{2} + A_{so.eff}\cdot\left(t_d + \frac{b_{so.eff}}{2}\right)\cdots \right. \\ &+2\cdot\left[t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2}\right)\cdots\right. \\ &\left.\left.+t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2}\right) + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2}\right)\right]\cdots\right. \\ &+A_w\cdot\left(t_d + b_s + \frac{b_{wy}}{2}\right) + A_{so}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(b_{wy} - s_{so0}) - \frac{b_{so.y}}{2}\right]\cdots \\ &\left.+A_b\cdot\left(b_{tb} - \frac{b_b}{2}\right)\right. \end{aligned}}{A_{tot.eff}} = 365.9\cdot mm$

$I_{y.s.eff} := 2\cdot\left[\frac{t_s\cdot b_{s.eff.c1}^{\,3}}{12} + t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2} - z_{sp.eff.neg}\right)^2\cdots\right.$
$\left.+\frac{t_s\cdot b_{s.eff.c2}^{\,3}}{12} + t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2} - z_{sp.eff.neg}\right)^2\cdots\right.$
$\left.+\frac{t_s\cdot b_{s.t}^{\,3}}{12} + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2} - z_{sp.eff.neg}\right)^2\right] = 3.609\times10^{-4}\cdot m^4$

$I_{A.c.eff.loc} := 5.316\times10^{3}\cdot mm^2\cdot s_{so}\cdot t_d\cdot n_{sd.eff}\cdot\left(\frac{t_d}{2} - z_{sp.eff.neg}\right)^2 = 4.176\times10^{-3}\cdot m^4$

$A_{c.eff.loc.mod} := \frac{\rho_c\cdot f_y(t_d)\cdot A_{sl}}{\sigma_{com.Ed}\cdot\gamma_M} = -5.379\times10^{3}\cdot mm^2$

$I_{y.tot.neg} := I_{y.so.eff} + I_{y.d.eff} + I_{y.w.eff} + I_{y.sl.eff} + I_{y.d.eff} = 0.015\cdot m^4$

$A_{sd.eff.neg} := A_b$ Bottom flange in tension

Effective cross section with negative moment(lower flange in tension)
The effective area in compression is calculated according to 4.5.2(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

Area

$A_{tot.eff} := \begin{cases} \rho_c\cdot A_{c.eff.loc.mod} + s_{so.eff}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot A_{c.eff.loc} + s_{so.eff}\cdot t_d & \text{otherwise} \end{cases} = 456.506\cdot cm^2$

A_{so} Lower stiffener in tension -> no reduction

A_{bt} Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.loc} := A_{c.eff} + A_{s.eff} + A_w + A_{so} + A_b = 1.118\times10^{-3}\cdot m^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c\cdot\frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot t_d & \end{cases} = 7.985\cdot mm$

$z_{sp.eff.neg} := \frac{\begin{aligned} &n_{so}\cdot b_{so.eff}\cdot s_{so}\cdot\frac{t_{d.eff}}{t_d}\cdot\left(t_d + \frac{b_{so}}{2}\right) + s_{so.eff}\cdot t_d\cdot\frac{t_d}{2}\cdots \\ &+n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_d}{2}\right) + t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2}\right)\cdots \\ &+t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2}\right) + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2}\right)\cdots \\ &+A_w\cdot\left(t_d + b_s + \frac{b_{wy}}{2}\right) + A_{so}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(b_{wy} - s_{so0}) - \frac{b_{so.y}}{2}\right]\cdots \\ &+A_b\cdot\left(b_{tb} - \frac{b_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 383.576\cdot mm$

Effective moment of inertia:

$I_{y.d.eff.lin} := \frac{n_{sd}\cdot s_{so.eff}\cdot t_d^{\,3}}{12} + t_d\cdot n_{sd}\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{sp.eff.neg}\right)^2 = 4.595\times10^{-3}\cdot m^4$

$z_{so0} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{so0} = 7.206\cdot mm$

Stiffeners overlunder:

$I_{y.st.so.fin} := n_{so}\cdot\frac{t_{so}\cdot b_{so.eff}^{\,3}}{12} + n_{so}\cdot t_{so}\cdot b_{so.eff}\cdot\left(t_d + \frac{b_{so.eff}}{2} - z_{sp.eff.neg}\right)^2 = 2.127\times10^{-3}\cdot m^4$

$I_{y.st.su.1.fin} := 6.1.506\cdot10^{-6}\cdot m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 8.154\times10^{-4}\cdot m^4$
$+2\cdot t_{so}\cdot b_{so}\cdot\left[t_d + b_s + \frac{1}{3}\cdot(b_{wy} - s_{so0y}) - \frac{b_{so.y}}{2} - z_{sp.eff.neg}\right]^2\cdots$
$+2\cdot t_{so}\cdot b_{so}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(b_{wy} - s_{so0y}) - \frac{b_{so.y}}{2} - z_{sp.eff.neg}\right]^2\cdots$
$+2\cdot t_{so}\cdot b_{so}\cdot\left[t_d + b_s + (b_{wy} - s_{so0y}) - \frac{b_{so.y}}{2} - z_{sp.eff.neg}\right]^2$

Web:

$I_{y.w.1.fin} := 2\cdot0.000312\cdot m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2\cdot t_w\cdot b_w\cdot\left[b_{tb} - \frac{b_b}{2} - \frac{1}{2}\cdot(b_{wy}) - z_{sp.eff.neg}\right]^2 = 2.749\cdot10^{-3}\cdot m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + b_b \cdot t_b \cdot \left(h_n - z_{sp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.975 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{sp.eff.neg}\right)^2 \dots \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{sp.eff.neg}\right)^2 \dots \\ + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{sp.eff.neg}\right)^2 \end{array}\right] = 4.171 \times 10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.468 \times 10^{-2}\ m^4$

Load

$q_M := 95 \frac{kN}{m^2}$

$Q_M_{bridge} := q_M \cdot b_d = 332.5 \frac{kN}{m}$ $V_{Ed} := \frac{Q_M_{bridge}}{2} = 2.993 \times 10^3\ kN$

$M_{Ed} := \frac{Q_M_{bridge} \cdot l_1^2}{8} = 1.347 \times 10^4\ kN \cdot m$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{sp.eff.neg}} = 0.038\ m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.359 \times 10^4\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.991$

Shear capacity

$\eta_1 := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot t(t_w) \cdot t_w = 0.293\ m$

$\alpha_w := \asin\left(\frac{h_{wx}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_b}{2} + z_{sd0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} \cdot \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right)}{2} \cdot h_{sw} = 0.503\ m$

$a_{st.z.2} := a_{st.z.1} + h_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\ m$

$a_{st.z.3} := a_{st.z.2} + h_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\ m$

Moment of inirta stiffeners z-direction

$I_{st.z} := 2 \cdot t_{sw} \cdot h_{sw} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\ m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$z_{sweb} := \frac{h_b}{2} + z_{sd0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.525\ m$

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{sti} = 0$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot t(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.455$ $I_{st.z.tot} := I_{st.z} + I_{sw.z} = 0.029\ m^4$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

Shear buckling coefficient, side plate

$\frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau s1} := \max\left[0, \left[\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}} - \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right]\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4 \cdot \left(\frac{h_{web}}{a}\right)^2 + k_{\tau s1} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}} \cdot \alpha^2 + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\lambda_w = 1.2$ EC1993-1-3-5 ANM.2 5.1

$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon(t_s)}{\eta_1} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} =$ "Reduce"

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$ SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta_1 & \text{if } \lambda_w < \frac{0.83}{\eta_1} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta_1} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ SS-EN 1993-1-5 5.3 Table 5.1, stiff end stiffener.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M1} = 1.00$

SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3\ kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.993 \times 10^3\ kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.704$ Contribution from two web

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear bucking --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{sd.eff.neg} := z_{sp.eff.neg} - \frac{t_d}{2} = 378.576\ mm$

$z_{sd.eff.neg} := h_n - \frac{h_b}{2} - z_{sp.eff.neg} = 498.424\ mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{sd.eff.neg} \cdot \left|z_{sd.eff.neg}\right| \cdot f_y(t_d)}{\gamma_{M0}} \dots = 7.14 \times 10^3\ kN \cdot m$
$+ \frac{A_{sd.eff.neg} \cdot \left|z_{sd.eff.neg}\right| \cdot f_y(t_s)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{sf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.623\ m$

$c_{sf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.59\ m$

$V_{bf.Rd.cored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{sf} \cdot \gamma_{M1} \cdot \gamma_n} \dots \frac{b_b \cdot \eta^2 \cdot f_y(t_b)}{c_{sf} \cdot \gamma_{M1} \cdot \gamma_n} = 294.676\ kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} = 0\ kN \\ 0 & \text{otherwise} \end{cases}$

Shear capacity stiffened web:

$\frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3\ kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.704$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.57 \times 10^{-3}\ m^4$

The flanges effective area:

$A_{eff.f.tot} := n_{so.eff} \cdot t_d + n_{sc} \cdot n_{so.eff} \cdot t_{d.eff} + n_{sc} \cdot n_{sx} \cdot t_{d.eff} + A_b = 435.406\ cm^2$

$z_{sp.f.eff.neg} := \frac{\begin{array}{l} n_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{sc} \cdot n_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \dots \\ + n_{sc} \cdot n_{sx} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_n - \frac{t_b}{2}\right) \end{array}}{A_{eff.f.tot}} = 326.711\ mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{sp.f.eff.neg}} = 0.026\ m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 9.313 \times 10^3\ kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.359 \times 10^4\ kN \cdot m$

Moment capacity check when $q_M := 95 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.991$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The reduced stress method

-Changed top plate stiffener thickness to 18 mm

Material

$f_y(t) = \begin{cases} 355\,MPa & \text{if } t < 40\,mm \\ 335\,MPa & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$E = 210\,GPa$

$\nu = 0.3$ $G = \frac{E}{2(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$t_b = 16mm$ $b_b = 1000mm$ $A_b = b_b t_b = 0.016m^2$

Top plate

$t_{tp} = 10mm$ $b_{tp} = 3500mm$ $A_{tp} = b_{tp} t_{tp} = 0.035m^2$

Side plates

$t_s = 12mm$ $h_s = 389.2mm$ $A_s = h_s t_s = 4.67 \cdot 10^{-3} m^2$

Webs

$t_w = 12mm$ $h_w = 1340mm$ $A_w = h_w t_w = 0.016m^2$

$h_{w,n} = 1250mm$ $h_{w,p} = 482.8mm$

Top stiffeners

$t_{st} = 18mm$ $b_{st} = 140mm$ $A_{st} = b_{st} t_{st} = 2.52 \cdot 10^{-3} m^2$

$s_{st} = 350mm$ $n_{st} = 9$

Web stiffeners

$t_{wst} = 12mm$ $b_{wst} = 120mm$ $A_{wst} = b_{wst} t_{wst} = 1.44 \cdot 10^{-3} m^2$

$s_{wst} = 440mm$ $n_{wst} = 3$

Whole bridge

$t_{bridge} = 18m$ $h_b = \frac{h_s}{2} + h_{w,p} + h_s + t_{tp} = 0.89m$ $n_b = 2m$

$I_{yb} = 0.015802m^4$ From grasshopper

$y_{tp} = 0.532713m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_z = 82.\frac{kN}{m^2}$

$Q = q \cdot n_b = 287.\frac{kN}{m}$

$M_{Ed} = \frac{Q \cdot t_{bridge}^2}{8} = 1.162 \cdot 10^4\,kN\,m$

$V_{Ed} = \frac{Q \cdot t_{bridge}}{2} = 2.583 \cdot 10^3\,kN$

Top plate

$\sigma_{x,Ed,tp} = \frac{M_{Ed}}{I_{yb}} \left(h_b - \frac{t_{tp}}{2} - y_{tp} \right) = 272.951\,MPa$ $\sigma_{z,Ed,tp} = 0$

$S_{tp} = \frac{t_{tp} b_{tp}}{n_{st}+1} \left(h_b - \frac{t_{tp}}{2} - y_{tp} \right) = 1.121 \cdot 10^{-3} m^3$

$\tau_{Ed,tp} = \frac{V_{Ed} S_{tp}}{I_{yb} \min(t_{tp}, t_{st})} = 16.083\,MPa$

Side plates

$\sigma_{x,Ed,sup} = \frac{M_{Ed}}{I_{yb}} \left(h_b - t_{tp} - y_{tp} \right) = 269.077\,MPa$

$\sigma_{z,Ed,sp} = \frac{Q}{n_s} = 11.958\,MPa$

$S_{sp} = \left(h_b - t_{tp} - y_{tp} \right) \left(t_s \frac{h_b - t_{tp} - y_{tp}}{2} + \frac{t_{tp} b_{tp}}{n_{st}+1} \left(h_b - \frac{t_{tp}}{2} - y_{tp} \right) \right) = 0.01\,m^3$

$S_{tps} = \frac{\left(t_{st} \frac{t_{st} n_{st}}{2} + 2 \frac{t_s b_{st}}{2} \right) \left(h_b - t_{tp} - \frac{h_{st}}{4} - y_{tp} \right)}{n_{st}+2} + \frac{t_s b_{st}}{n_{st}+2} \left(h_b - \frac{t_{tp}}{2} - y_{tp} \right) = 1.491 \cdot 10^{-3} m^3$

Top plate stiffeners

$\tau_{Ed,tps} = \frac{V_{Ed} S_{tps}}{I_{yb} t_{st}} = 14.258\,MPa$ $269.077\,MPa$

Governing slenderness from FE-software

$\sigma_{y,UNIT} = 4.34\frac{MPa}{\frac{mm}{m}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_y = \sigma_{y,UNIT} q = 355.88\,MPa$

$\alpha_{ult} = \frac{f_y(t_b)}{\sigma_y} = 0.998$

$\sigma_{cr,UNIT} = 245.28\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} = \frac{\sigma_{cr,UNIT}}{q} = 2.991$

$\lambda_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.577$ Slenderness for all parts

Top plate check

$\psi_{tp} = 1$

$\rho_{p,tp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E,tp} = \frac{\pi^2 E t_{tp}^2}{12(1-\nu^2) b_{tp}^2} = 1.549\,MPa$

$I_{y,tp} = \frac{b_{tp} t_{tp}^3}{12(1-\nu^2)} = 3.205 \cdot 10^{-7} m^4$

$y_{tp,sup} = \frac{A_{tp} \frac{t_{tp}}{2} + n_{st} A_{st} \left(t_{tp} + \frac{h_{st}}{2} \right)}{A_{tp} + n_{st} A_{st}} = 34.49\,mm$

$I_{yt,tp} = \frac{b_{tp} t_{tp}^3}{12} + A_{tp} \left(y_{tp,sup} - \frac{t_{tp}}{2} \right)^2 + n_{st} \left[\frac{t_{st} h_{st}^3}{12} + A_{st} \left(t_{tp} + \frac{h_{st}}{2} - y_{tp,sup} \right)^2 \right] = 1.147 \cdot 10^{-5} mm^4$

$\gamma_{tp} = \frac{I_{yb}}{I_{yt,tp}} = 358.013$

$\delta_{tp} = \frac{n_{st} A_{st}}{A_{tp}} = 0.648$

$\alpha_{tp} = \max\left(\frac{b_{tp}}{t_{tp}}, 0.5 \right) = 0.571$

$k_{\sigma,p,tp} = \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2 \right)^2 \left(1+\delta_{tp} \right) + \gamma_{tp} - 1 \right]}{\alpha_{tp}^2 \left(\alpha_{tp} + 1 \right)^2 \left(1+\delta_{tp} \right)} & \text{if } \alpha_{tp} < \sqrt{\gamma_{tp}} = 666.712 \\ \frac{4\left(1+\sqrt{\gamma_{tp}} \right)}{\left(\alpha_{tp}+1 \right)^2 \left(1+\delta_{tp} \right)} & \text{if } \alpha_{tp} \ge \sqrt{\gamma_{tp}} \end{cases}$

$\sigma_{cr,p,tp} = k_{\sigma,p,tp} \sigma_{E,tp} = 1.033 \cdot 10^3\,MPa$

Column-like behaviour [A.5.3]

$b_{1,sup,tp} = \frac{3}{5+\psi_{tp}} b_{tp} = 0.175m$

$b_{2,sup,tp} = \frac{2}{5-\psi_{tp}} b_{tp} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{d,1,tp} = b_{1,sup,tp} + b_{2,sup,tp} = 0.35m$

$y_{tp,cl,tp} = \frac{t_{tp} b_{d,1,tp} \frac{t_{tp}}{2} + n_{st} \left[A_{st} \left(t_{tp} + \frac{h_{st}}{2} \right) \right]}{t_{tp} b_{d,1,tp} + n_{st} A_{st}} = 0.036m$ combined center of gravity for stiffener and contributing top plate.

$I_{yt,1,tp} = \frac{b_{d,1,tp} t_{tp}^3}{12} + b_{d,1,tp} t_{tp} \left(y_{tp,cl,1,tp} - \frac{t_{tp}}{2} \right)^2 + \frac{t_{st} h_{st}^3}{12} + A_{st} \left(t_{tp} + \frac{h_{st}}{2} - y_{tp,cl,1,tp} \right)^2$

Combined moment of inertia

$\sigma_{cr,cl,tp} = \frac{\pi^2 E t_{tp,1,tp}^2}{12(1-\nu^2) b_{d,1,tp}^2} = 1.066 \cdot 10^3\,MPa$

Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,p} = \sigma_{cr,cl}$

$\sigma_{cr,c,tp} = \sigma_{cr,cl,tp}$

Reduction factor for column-like behaviour

$\gamma_{tp} = \sqrt{\frac{I_{yt,1,tp}}{A_{d,1,tp}}} = 0.045m$

$e_{tp} = \max\left(y_{tp,cl,1,tp} - t_{tp}/2 + \frac{h_{st}}{2} - y_{tp,cl,1,tp} \right) = 0.044m$

$\alpha_{e,tp} = 0.49 + \frac{0.09}{\frac{e_{tp}}{\gamma_{tp}}} = 0.577$ Stiffener with open cross section (curve c)

$\phi_{tp} = 0.5\left[1 + \alpha_{e,tp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 0.776$ [6.3.1.2]

$\chi_{tp} = \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1 \right) = 0.773$

Interpolation

$\psi_{tp} = \min\left(\max\left(\frac{\sigma_{cr,p,tp}}{\sigma_{cr,c,tp}} - 1.0 \right), 1 \right) = 0$

$\rho_{c,tp} = \begin{cases} \left[\left(\rho_{p,tp} - \chi_{tp} \right) b_{tp} \left(2 - t_{tp} \right) - \chi_{tp} \right] \frac{\left[\left(\rho_{p,tp} - \chi_{tp} \right) b_{tp} \left(2 - t_{tp} \right) + \chi_{tp} \right] n_{st} A_{st}}{\gamma_{M1}} < 1 & = 0.773 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp} \right) b_{tp} \left(2 - t_{tp} \right) + \chi_{tp} \right] n_{st} A_{st}}{\gamma_{M1}} > 1 \end{cases}$

Shear

$\eta_1 = 1.2$ 7.1 (2)

$\lambda_w = \begin{cases} 0 & \text{if } \lambda_p < \frac{0.83}{\eta_1} = 1.2 \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_1} < \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p > 1.08 \end{cases}$

$\frac{\rho_{c,tp} \alpha_{ult}}{\gamma_{M1}} = 0.771$

Side plates check

x-direction

$\sigma_{max} = f_y(t_s)$

$\sigma(z) = \left[\frac{\sigma_{max}}{f_y(t_s)} \cdot z + \sigma_{max} \right] \cdot 1$

$\psi_{sp,x} = \frac{\sigma\left(h_b - t_{tp} - h_s \right)}{\sigma\left(h_b - t_{tp} \right)} = -0.121$

$\sigma_{x,sup} = \frac{M_{Ed}}{I_{yb}} \left(h_b - t_{tp} - y_{tp} \right) = 269.077\,MPa$

$\sigma_{x,hst} = \frac{M_{Ed}}{I_{yb}} \left(h_b - t_{tp} - h_s - y_{tp} \right) = -32.474\,MPa$

$\psi_{sp,k,x} = \max\left(3 \cdot \psi_{sp,x}, 0 \right) = 2.879$ $\psi_{sp,x} = \frac{\sigma_{x,hst}}{\sigma_{x,sup}} = -0.121$

$\rho_{p,sp,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{0.2}{\left(1.05 + \psi_{sp,k} \right)} & \text{if } 1 > \psi_{sp,k} > 0 \\ 7.81 & \text{if } \psi_{sp,k} = 0 \\ 7.81 - 6.29\,\psi_{sp,k} + 9.78\,\psi_{sp,k}^2 & \text{if } 0 > \psi_{sp,k} > -1 \\ 23.9 & \text{if } \psi_{sp,k} = -1 \\ 5.98\left(1 - \psi_{sp,k} \right)^2 & \text{if } -1 > \psi_{sp,k} > -3 \\ 95.68 & \text{if } \psi_{sp,k} < -3 \end{cases} = 1$

$k_{\sigma,sp,x} = \begin{cases} 1 & \text{if } \psi_{sp,x} \ge 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp,x} \right)} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\,\psi_{sp,x} + 9.78\,\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98\left(1 - \psi_{sp,x} \right)^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 95.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 8.712$

$\sigma_{cr,sp,sp,x} = \sigma_{E,sp,x} k_{\sigma,sp,x} = 1.572 \cdot 10^3\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,x} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) b_{sp}^2} = 6.833\,MPa$

$\alpha_{sp} = 0.21$

$\phi_{sp} = 0.5\left[1 + \alpha_{sp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 0.706$

$\chi_{sp} = \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1 \right) = 0.898$

Interpolation

$\psi_{sp,x} = \min\left(\max\left(\frac{\sigma_{cr,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0 \right), 1 \right) = 1$

$\rho_{c,sp,x} = \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp} \right) b_{sp,x} \left(2 - t_{sp,x} \right) - \chi_{sp} \right] \frac{\left[\left(\rho_{p,sp} - \chi_{sp} \right) b_{sp,x} \left(2 - t_{sp,x} \right) - \chi_{sp} \right] n_{sst} A_{sst}}{\gamma_{M1}} < 1 & = 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp} \right) b_{sp,x} \left(2 - t_{sp,x} \right) - \chi_{sp} \right] n_{sst} A_{sst}}{\gamma_{M1}} > 1 \end{cases}$

Z-direction

$\psi_{sp,z} = 1$

$\alpha_{sp,sp} = 0.34$ $b_{sp,sp} = 0.7$

$\phi_{p,sp,sp} = 0.5\left[1 + \alpha_{sp,sp} \left(b_{sp,sp} - 0.2 \right) + \lambda_p \right] = 0.768$

$\rho_{p,sp,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{p,sp} + \sqrt{\phi_{p,sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,p,sp,z} = 4$

$\sigma_{E,sp,z} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) b_{sp}^2} = 6.833\,MPa$

$\sigma_{cr,p,sp,z} = k_{\sigma,p,sp,z} \sigma_{E,sp,z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,z} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) b_s^2} = 180.432\,MPa$

Interpolation

$\psi_{sp,z} = \min\left(\max\left(\frac{\sigma_{cr,p,sp,z}}{\sigma_{cr,c,sp,z}} - 1.0 \right), 1 \right) = 0$

$\rho_{c,sp,z} = \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp} \right) b_{sp,z} \left(2 - t_{sp,z} \right) - \chi_{sp} \right] \frac{\left[\left(\rho_{p,sp} - \chi_{sp} \right) b_{sp,z} \left(2 - t_{sp,z} \right) - \chi_{sp} \right] n_{sst} A_{sst}}{\gamma_{M1}} < 1 & = 0.898 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp} \right) b_{sp,z} \left(2 - t_{sp,z} \right) - \chi_{sp} \right] n_{sst} A_{sst}}{\gamma_{M1}} > 1 \end{cases}$

Top plate stiffeners - check

x-direction

$\rho_{p,sp,sst} = \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$$\psi_{fps,x} := \frac{d\left(h_{fx} - t_f - h_{sh}\right)}{d\left(h_{fx} - t_f\right)} = 0.597$$

$$\sigma_{E,fps,x} := \frac{\pi^2 \cdot 6 \cdot k_{fps}^{\ 2}}{12\left(1 - \nu^2\right)\left(\frac{2t_f}{3}\right)^2 \cdot b_{fx}^{\ 2}} = 3.138 \cdot 10^3 \cdot \text{MPa}$$

$$k_{\sigma,fps,x} := \begin{cases} 1 & \text{if } \psi_{fps,x} \geq 1 \\ \frac{8.2}{\left(1.05 + \psi_{fps,x}\right)} & \text{if } 1 > \psi_{fps,x} > 0 \\ 7.81 & \text{if } \psi_{fps,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{fps,x} + 9.78 \cdot \psi_{fps,x}^{\ 2} & \text{if } 0 > \psi_{fps,x} > -1 \\ 23.9 & \text{if } \psi_{fps,x} = -1 \\ 5.98 \cdot \left(1 - \psi_{fps,x}\right)^2 & \text{if } -1 > \psi_{fps,x} > -3 \\ 95.68 & \text{if } \psi_{fps,x} < -3 \end{cases} = 4.198$$

$$\sigma_{cr,fps,x} := k_{\sigma,fps,x} \cdot \sigma_{E,fps,x} = 1.317 \cdot 10^4 \cdot \text{MPa}$$

Column-like behaviour

$$\sigma_{cr,c,fps,x} := \frac{\pi^2 \cdot 6 \cdot k_{fps}^{\ 2}}{12\left(1 - \nu^2\right)\left(\frac{2t_f}{3}\right)^2 \cdot b_{fx}^{\ 2}} = 15.374 \cdot \text{MPa}$$

$$\chi_{fps} := \chi_{ip}$$

Interpolation

$$\xi_{fps,x} := \min\left(\max\left(\frac{\sigma_{cr,fps,x}}{\sigma_{cr,c,fps,x}} - 1.0\right), 1\right) = 1$$

$$\rho_{c,fps,x} := \begin{cases} \left[\left(\rho_{fps,ip} - \chi_{ip}\right)\xi_{fps,x}\left(2 - \xi_{fps,x}\right) - \right] \cdot \chi_{ip} & \text{if } \frac{\left[\left(\rho_{fps,ip} - \chi_{ip}\right)\xi_{fps,x}\left(2 - \xi_{fps,x}\right) - \right] \cdot \chi_{ip}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{fps,ip} - \chi_{ip}\right)\xi_{fps,x}\left(2 - \xi_{fps,x}\right) + \chi_{ip}\right] \cdot \alpha_{fx}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,ip}}{\frac{\rho_{c,fps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,ip}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.993$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,ip}}{\frac{\rho_{c,fps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,ip}}{\frac{\rho_{c,wb,x} \cdot f_y}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,ip}}{\frac{\rho_{c,fps,x} \cdot f_y}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z,Ed,ip}}{\frac{\rho_{c,wb,x} \cdot f_y}{\gamma_{M1}}}\right) + 3\left(\frac{\tau_{Ed,ip}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.633$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,sps}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,sps}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.578$$

C.4 Changed side plate thickness

CCXXXI 2 mm side plate EWM calculation.
CCXXXIV 1 mm side plate RSM calculation.
CCXXXVI 3 mm side plate EWM calculation.
CCXXXIX 3 mm side plate RSM calculation.
CCXLI 4 mm side plate EWM calculation.
CCXLIV 4 mm side plate RSM calculation.
CCXLVI 6 mm side plate RSM calculation.
CCXLIX 6 mm side plate EWM calculation.
CCLI 8 mm side plate RSM calculation.
CCLIV 8 mm side plate EWM calculation.
CCLVI 10 mm side plate RSM calculation.
CCLIX 10 mm side plate EWM calculation.
CCLXI 14 mm side plate RSM calculation.
CCLXIV 14 mm side plate EWM calculation.

EWM - Changed side plate thickness

This chapter covers the EWM when changing the side plate thickness to 2 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s,sp} := n_{su} + 1 = 10$

Deck: $b_d := 3500\text{mm}$ $t_d := 10\text{mm}$

$b_{d,0.5} := 1750\text{mm}$

$s_{su} := \frac{b_d}{n_{s,su}} = 350\text{ mm}$

Side plate: $h_s := 389.2\text{mm}$ $t_s := 2\text{mm}$

Web: $h_w := 1340\text{mm}$ $t_w := 12\text{mm}$

$h_{wy} := 482.8\text{mm}$

$h_{wx} := 1250\text{mm}$

$s_{su0} := 20\text{mm}$

$s_{su} := 440\text{mm}$

Bottom plate: $b_b := 1000\text{mm}$ $t_b := 16\text{mm}$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140\text{mm}$ $t_{su} := 16\text{mm}$ $a_{su} := 6\text{mm}$

Stiffeners under: $h_{su} := 120\text{mm}$ $t_{su} := 12\text{mm}$ $a_{su} := 5\text{mm}$

$h_{su,y} := h_{su}\cdot\cos\left(21.12\cdot\frac{\pi}{180}\right) = 111.939\text{ mm}$

Total height: $h_{tx} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890\text{ mm}$

Bridge length: $l_{bridge} := 18\text{m}$

Distance between vertical stiffeners: $a := 2\text{m}$

Transverse stiffeners: $s_{ts} := 2000\text{mm}$ $s_{ts0} := 500\text{mm}$

$t_{ts.1} := 18\text{mm}$ $t_{ts2} := 12\text{mm}$ $t_{ts3} := 8\text{mm}$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864\text{ mm}$

Poissons ratio $\upsilon := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d\cdot b_d$ **parameter:** $A_s := 2t_s\cdot h_s$ $A_w := 2t_w\cdot h_w$ $A_b := t_b\cdot b_b$

$A_{su} := n_{su}\cdot t_{su}\cdot b_{su}$ $A_{sl} := n_{sl}\cdot t_{sl}\cdot b_{su}$

$E := 210\text{GPa}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.114\,\text{m}^2$ tab. 3.1

$A_c := A_{su} + A_d = 0.055\,\text{m}^2$ up

$\varepsilon(t) := \sqrt{\frac{235\text{MPa}}{f_y(t)}}$

Change of center of gravity, z_{sp}

(From top to centre of gravity)

$$z_{sp} := \frac{A_d\cdot\frac{t_d}{2} + A_{su}\cdot\left(t_d + \frac{h_{su}}{2}\right) + A_s\cdot\left(t_d + \frac{h_s}{2}\right) + A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + \ldots}{A_{tot}} = 373.965\text{ mm}$$

$$+ A_b\cdot(t_d + h_s + h_{wy}) + A_{sl}\cdot\left[t_d + h_s + \frac{2}{3}\cdot(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\text{ mm}$

Stiffeners over/under:

$$I_{y.st.su} := n_{su}\cdot\frac{t_{su}\cdot h_{su}^3}{12} + n_{su}\cdot t_{su}\cdot h_{su}\cdot\left(t_d + \frac{h_{su}}{2} - z_{sp}\right)^2 = 1.775\times10^{-3}\,\text{m}^4$$

$I_{y.st.su.1} := 6\cdot1.506\cdot10^{-6}\,\text{m}^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su}\cdot h_{su}\left[t_d + h_s + \frac{1}{3}\cdot(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \ldots = 8.621\times10^{-4}\,\text{m}^4$$

$$+ 2t_{su}\cdot h_{su}\left[t_d + h_s + \frac{2}{3}\cdot(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \ldots$$

$$+ 2t_{su}\cdot h_{su}\left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{sp}\right]^2$$

Web:

$I_{y.w.1} := 2\cdot0.00312\text{m}^4$ from FEM

$$I_{y.w} := I_{y.w.1} + 2t_w\cdot h_w\left[h_{tx} - \frac{h_b}{2} - \frac{1}{2}\cdot(h_{wy}) - z_{sp}\right]^2 = 2.91\times10^{-3}\,\text{m}^4$$

Deck:

$$I_{y.d} := \frac{b_d\cdot t_d^3}{12} + t_d\cdot b_d\cdot\left(\frac{t_d}{2} - z_{sp}\right)^2 = 4.765\times10^{-7}\,\text{m}^4$$

Bottom flange:

$$I_{y.b} := \frac{b_b\cdot t_b^3}{12} + t_b\cdot b_b\cdot\left(h_{tx} - z_{sp} - \frac{t_b}{2}\right)^2 = 4.13\times10^{-3}\,\text{m}^4$$

Side plate:

$$I_{y.s} := 2\cdot\frac{t_s\cdot h_s^3}{12} + 2t_s\cdot h_s\cdot\left(t_d + \frac{h_s}{2} - z_{sp}\right)^2 = 6.431\times10^{-5}\,\text{m}^4$$

Total:

$$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,\text{m}^4$$

Reduced cross section

Functions to investigate the stresses

Stresses

$\sigma_{max} := f_y(t_d) = 355\text{ MPa}$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{sp}}\cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{sp}}\cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi \equiv 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi \equiv 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi \equiv -1 \\ 5.98\cdot(1-\psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ 0.57 & \text{if } \psi \equiv 0 \\ 0.85 & \text{if } \psi \equiv -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi \equiv 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi \equiv -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(b_p, t, \varepsilon) := \frac{\frac{b_p}{t}}{28.4\cdot\varepsilon(t)\cdot\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\cdot\max(3+\psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\text{ mm}$

$z_2 := t_d + h_s = 399.2\text{ mm}$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.069$

$$\text{Condition} := \begin{cases} 62\cdot\varepsilon(t_s)\cdot(1-\psi_s)\cdot\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42\cdot\varepsilon(t_s)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$$

$$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.293$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(h_{\sigma.4.1}(\psi_s), h_s, t_s) = 2.924$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 0.323$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{sp} = 0.025\text{ m}$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 363.965\text{ mm}$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 117.596\text{ mm}$

$h_{s.eff.c1} := 0.4\cdot h_{s.eff.c} = 47.038\text{ mm}$

$h_{s.eff.c2} := 0.6\cdot h_{s.eff.c} = 70.557\text{ mm}$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 142.831\text{ mm}$

$A_{s.eff.c} := 2\cdot h_{s.eff.c}\cdot t_s = 470.383\text{ mm}^2$

$A_{s.eff} := 2\cdot(h_{s.eff.c} + h_{s.t})\cdot t_s = 571.325\text{ mm}^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_d)\frac{E\cdot\pi^2\cdot t_s^2}{12\cdot(1-\nu^2)\cdot b_w^2} = 41.565\cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$h_w := t_d = 10\cdot mm$

$b_w := t_d + b_{so} = 150\cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.615$

$k_{\sigma.4.2}(\psi_{so}) = 0.605$ Higher stress close to support

$Class4_s := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{so}}{t_{so}} > 21\cdot\varepsilon\cdot\left(\sqrt{k_{\sigma.4.2}(\psi_{so})}\right) \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2}(\psi_{so}), b_{so}, t_{so}) = 0.487$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_d) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so}\cdot b_{so} = 140\cdot mm$

$A_{so.eff} := n_{so}\cdot t_{so}\cdot b_{so.eff} = 2.016\times 10^{3}\cdot mm^2$

Deck

$\frac{b_{so}}{t_d} = 35 \qquad 42\sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_s := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{so}}{t_d} > 42\sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5\times 10^{-3}\cdot m \qquad \psi_d := \psi_{neg}(z_d, \tau_d) = 1$

$n_{so} := t_{so} \quad$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, t_{cao}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$t_{so.eff} := \rho_d\cdot t_{cao} = 321.461\cdot mm$

Areas:

$A_{so} := n_{so}\cdot t_{so}\cdot t_d + n_{so}\cdot t_{so}\cdot(b_{so.eff} + t_d) = 5.31\times 10^{4}\cdot mm^2$

$A_{c.eff.loc} := n_{so}\cdot t_{so}\cdot t_d + n_{so}\cdot t_{so}\cdot(b_{so.eff} + t_d) = 5.053\times 10^{4}\cdot mm^2$

More than three longitudinal stiffeners -> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times 10^{-5}\cdot m^4$

$y_{tp.sl} := \frac{n_{so}\cdot t_{so}\cdot\left(t_d + \frac{b_{so}}{2}\right) + b_d\cdot t_d\cdot\frac{t_d}{2}}{n_{so}\cdot t_{so}\cdot b_{so} + b_d\cdot t_d} = 32.411\cdot mm$

$I_{sl} := \left[\frac{b_d\cdot t_d^3}{12} + b_d\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] + \left[\frac{n_{so}\cdot t_{so}\cdot b_{so}^3}{12} + n_{so}\cdot b_{so}\cdot t_{so}\cdot\left(t_d + \frac{b_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052\times 10^{-8}\cdot m^4$

$\beta_c := \frac{A_{sl}}{A_p} = 0.576 \qquad \alpha := \frac{t_s}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$k_{\sigma.p} := \begin{cases} \frac{2\cdot\left[(1+\alpha^2)^2 + \gamma - 1\right]}{\alpha^2\cdot(\psi+1)\cdot(1+\delta)} & \text{if } \alpha < \sqrt{\gamma} \\ \frac{4\cdot\left(1+\sqrt{\gamma}\right)}{(\psi+1)\cdot(1+\delta)} & \text{otherwise} \end{cases} = 639.127$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p}\frac{E\cdot\pi^2\cdot t_s^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 990.256\cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{tp} := \sqrt{\frac{\beta_{A.c}\cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi)\cdot A_{c.eff.loc} + t_{so.eff}\cdot t_d = 5.375\times 10^{4}\cdot mm^2$

Bucking strength

$N_{Rd} := A_{c.eff}\frac{f_y(t_d)}{\gamma_M} = 1.908\times 10^{4}\cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so}\cdot b_{so}\cdot\left(t_d + \frac{b_{so}}{2}\right) + n_{so}\cdot t_d\cdot\frac{t_d}{2}}{t_{so}\cdot b_{so} + n_{so}\cdot t_d} = 34.268\cdot mm$

$I_{sl.c} := \left[\frac{n_{so}\cdot t_d^3}{12} + n_{so}\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] + \left[\frac{t_{so}\cdot b_{so}^3}{12} + b_{so}\cdot t_{so}\cdot\left(t_d + \frac{b_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137\times 10^{-7}\cdot m^4$

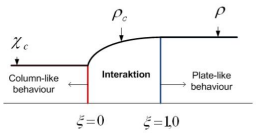
Bucking resistance stiffener

$A_{sl} := t_{so}\cdot b_{so} + n_{so}\cdot t_d = 5.74\times 10^{3}\cdot mm^2$

$A_{sl.eff} := t_{so}\cdot b_{so.eff} + n_{so.eff}\cdot t_d = 5.455\times 10^{3}\cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2\cdot E\cdot I_{sl.c}}{A_{sl}\cdot a^2} = 1.026\times 10^{3}\cdot MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c\cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\cdot mm$

$\bar{e}_c := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{b_{so}}{2} - y_{tp.sl}\right)\right] = 47.589\cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{\bar{e}_c}} = 0.522$

$\phi := 0.5\left[1 + \alpha_e\cdot(\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left[\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right] = 0$

ξ är nästan noll --> column like behvivour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c)\cdot\xi\cdot(2 - \xi) + \chi_c = 0.792$

Checks

Mean stress

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so}\cdot t_{so.eff}\cdot t_d\cdot\sigma_{neg}\left(t_d\cdot\frac{t_d}{2}\right) + A_{so}\cdot\sigma_{neg}\left(t_d + \frac{b_{so.eff}}{2}\right) = -1.576\times 10^{4}\cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so}\cdot t_{so.eff}\cdot t_d + A_{so}} = -321.016\cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c\cdot f_y(t_{so})}{\gamma_M} = 281.032\cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c\cdot f_y(t_d)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{so}\cdot t_{so}\cdot t_{so.eff} = 0.032\cdot m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{s.eff} = 0.11\cdot m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff}\cdot\frac{t_d}{2} + A_{so.eff}\cdot\left(t_d + \frac{b_{so.eff}}{2}\right) \dots \\ &+ 2\cdot\left[t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2}\right) \dots \right. \\ &\left. + t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2}\right) + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2}\right)\right] \dots \\ &+ A_w\cdot\left(t_d + b_s + \frac{h_{wy}}{2}\right) + A_{so}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(h_{wy} - t_{so.eff}) - \frac{h_{so.y}}{2}\right] \dots \\ &+ A_b\cdot\left(h_{tb} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 385.306\cdot mm$

$I_{y.s.eff} := 2\cdot\left[\frac{t_s\cdot b_{s.eff.c1}^3}{12} + t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right. \\ \left. + \frac{t_s\cdot b_{s.eff.c2}^3}{12} + t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2\right] = 2.406\times 10^{-7}\cdot m^4$

$A_{d.eff.neg} := \frac{A_{d.eff}}{2.754} + \frac{h_{s.t}}{t_s}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2} - z_{tp.eff.neg}\right)^2$

$A_{c.eff.loc} = 5.053\times 10^{4}\cdot mm^2$

$I_{A_{c.eff.loc.mod}} := \frac{\rho_c\cdot f_y(t_d)\cdot A_{sl}}{\sigma_{com.Ed}\cdot\gamma_M} = -5.025\times 10^{3}\cdot mm^2\cdot z_{tp.eff.neg}^2 = 4.65\times 10^{-3}\cdot m^4$ (A.3)

$I_{sl.eff.neg} := A_w\cdot\left[z_{tp.eff.neg} - t_d - b_s - \frac{h_{wy}}{2}\right]^2 + I_{so.eff.neg} + b_{so.eff}\cdot t_{so.eff}\cdot\left[t_d + b_s + \frac{t_{so.eff}}{2} - z_{tp.eff.neg}\right]^2 + I_{y.s.eff} + I_{y.d.eff} = 0.014\cdot m^4$

Bottom flange in tension

Reduction in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

Effective cross-section with negative moment(lower flange in tension)

$A_{c.eff.loc.mod} := \begin{cases} \rho_c\cdot A_{c.eff.loc.mod} + n_{so.eff}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot A_{c.eff.loc} + n_{so.eff}\cdot t_d & \text{otherwise} \end{cases} = 432.173\cdot cm^2$

A_{so} Lower stiffener in tension -> no reduction

A_{tb} Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_{so} + A_{tb} = 1.006\times 10^{-3}\cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c\cdot\frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot t_d & \end{cases} = 7.916\cdot mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so}\cdot b_{so.eff}\cdot t_{so}\cdot\frac{t_{d.eff}}{t_d}\cdot\left(t_d + \frac{b_{so}}{2}\right) + t_{so.eff}\cdot t_d\cdot\frac{t_d}{2} \dots \\ &+ n_{so}\cdot t_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_d}{2}\right) + t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2}\right) \dots \\ &+ t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2}\right) + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2}\right) \dots \\ &+ A_w\cdot\left(t_d + b_s + \frac{h_{wy}}{2}\right) + A_{so}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(h_{wy} - t_{so.eff}) - \frac{h_{so.y}}{2}\right] \dots \\ &+ A_b\cdot\left(h_{tb} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 415.78\cdot mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so}\cdot t_{so.eff}\cdot t_d^3}{12} + t_d\cdot n_{so}\cdot t_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.425\times 10^{-3}\cdot m^4$

$b_{edge} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot t_{so0} = 7.206\cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so}\cdot\frac{t_{so}\cdot t_{so.eff}^3}{12} + n_{so}\cdot t_{so}\cdot b_{so.eff}\cdot\left(t_d + \frac{b_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.306\times 10^{-3}\cdot m^4$

$I_{y.st.su.1.fin} := 6\cdot 1.506\cdot 10^{-5}\cdot m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots \\ + 2\cdot t_{so}\cdot b_{so}\cdot\left[t_d + b_s + \frac{1}{3}\cdot(h_{wy} - t_{so0}) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots \\ + 2\cdot t_{so}\cdot b_{so}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(h_{wy} - t_{so0}) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots \\ + 2\cdot t_{so}\cdot b_{so}\cdot\left[t_d + b_s + (h_{wy} - t_{so0}) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 = 6.704\cdot 10^{-4}\cdot m^4$

Web:

$I_{y.w.1.fin} := 2\cdot 0.000312\cdot m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2\cdot t_w\cdot b_w\cdot\left[b_s - \frac{t_b}{2} - \frac{1}{2}\cdot(h_{wy}) - z_{tp.eff.neg}\right]^2 = 2.249\times 10^{-7}\cdot m^4$

Bottom flange:

$I_{y,b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \left(h_n - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.478 \times 10^{-3}\,m^4$

Side plate:

$I_{y,s.fin} := 2\left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1}\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2}\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t}\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right]\right]\right] = 2.941 \times 10^{-5}\,m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.ss.fin} + I_{y.st.ns.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.416 \times 10^{-10}\,mm^4$

Load

Load capacity:

$q_{.M} := 85\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_{.M} := q_{.M} \cdot b_{tf} = 297.5\frac{kN}{m}$

$M_{Ed} := \frac{Q_{.M} \cdot l_{bridge}^2}{8} = 1.205 \times 10^4\,kN \cdot m$

$q_{.V} := 15\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_{.V} := q_{.V} \cdot b_{tf} = 52.5\frac{kN}{m}$

$V_{Ed} := \frac{Q_{.V} \cdot l_{bridge}}{2} = 472.5\,kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.034\,m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.209 \times 10^4\,kN \cdot m$

Moment capacity

$\frac{M_{Ed}}{M_{Rd}} = 0.997$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_{wb}) \cdot t_w = 0.293\,m$

$\alpha_w := \min\left(\frac{h_{ww}}{h_w}, \frac{180}{\pi}\right) = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.c.1} := \frac{h_b}{2} + x_{wb0} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) + \frac{t_{ns}}{2} - \frac{\sin\left(\alpha_w \frac{\pi}{180}\right) \cdot h_{ns}}{2} = 0.503\,m$

$a_{st.c.2} := a_{st.c.1} + x_{ns} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 0.913\,m$

$a_{st.c.3} := a_{st.c.2} + x_{ns} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 1.324\,m$

Moment of initia stiffeners z-direction

$I_{st.z} := 2\left[t_{ns} \cdot h_{ns}\left[\left(a_{st.c.1}\right)^2 + \left(a_{st.c.2}\right)^2 + \left(a_{st.c.3}\right)^2\right]\right] = 8.18 \times 10^{-3}\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{bw.z} = 0.029\,m^4$

Moment of inertia of stiffeners in the web area which is 15st each side of the stiffeners:

$z_{st} := a_{st.c.3} = 1.324\,m$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00\left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34\left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.36$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{sti} = 0$

Moment of inertia of 15st of the web z-direction

$I_{wz1} := \frac{h_{s.eff}}{37.4 \cdot t_w \cdot \varepsilon(t_w)\sqrt{k_{\tau 1}}} = 1.014$ EC3-1-5 A.3 (6)

Shear buckling coefficient, web with more than two longitudinal stiffeners

$\alpha_{sv} := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \sqrt{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_{wb}\right)^3 \cdot h_w\right)}\right]^3}, \frac{2.1}{\min\left(t_s, t_{wb}\right)} \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 2.694 \times 10^5$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18\frac{I_{st.z.tot}}{\min\left(t_s, t_{wb}\right)^3 h_{web}} + 2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_{wb}\right)^3 h_{web}}} & \text{otherwise} \end{cases} = 3.392 \times 10^5$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\lambda_{lim} := 1.2$ EC1993-1-3-5 ANM.2 5.1

$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_{wb}\right)} \leq 31\frac{\varepsilon(t_s)}{\eta}\sqrt{k_\tau} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 1.014$ Enligt SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 0.819$ SS-EN 1993-1-5 5.3 Table 5.1, stiff end stiffener.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M1} := 1.00$

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 241.66\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 472.5\,kN$

$\frac{V_{Ed}}{2V_{bw.Rd}} = 0.978$ (Contribution from two web)

$V_{b.Rd} = \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4, 583 bärande konstruktioner

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{tf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 410.78\,mm$

$a_{sf.eff.neg} := h_b - \frac{h_b}{2} - z_{tp.eff.neg} = 466.22\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \cdot \left|a_{tf.eff.neg}\right| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{sf.eff.neg} \cdot \left|a_{sf.eff.neg}\right| \cdot f_y(t_b)}{\gamma_{M0}} = 7.336 \times 10^2\,kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$\varepsilon_{tf} := \alpha\left[\frac{1.6 \cdot b_{tf} \cdot t_d^2 \cdot f_y(t_d)}{t_s\left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 1.236\,m$

$\varepsilon_{sf} := \alpha\left[\frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s\left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 1.039\,m$

$V_{bf.Rd.cered} := \frac{b_{tf} \cdot t_d^2 \cdot f_y(t_d)}{\varepsilon_{tf} \cdot \gamma_{M1} \cdot m} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{\varepsilon_{sf} \cdot \gamma_{M1} \cdot m} = 156.653\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cered}\left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} = 0\,kN \\ 0 & \text{otherwise} \end{cases}$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} = \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{bw.Rd} = V_{bw.Rd} + V_{bf.Rd} = 241.66\,kN$

Check:

$\frac{V_{Ed}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2V_{b.Rd}} = 0.978$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.903 \times 10^{-3}\,m^4$

The flanges effective area:

$A_{eff.f.tot} := n_{so.eff} \cdot t_d + n_{no} \cdot n_{so.eff} \cdot t_d.eff + n_{so} \cdot t_{so} \cdot t_d.eff + A_b = 432.579\,cm^2$

z_{tp} for the flanges effective areas:

$z_{tp.f.eff.neg} := \frac{n_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{no} \cdot n_{so.eff} \cdot t_d.eff\left(\frac{t_d.eff}{2}\right) + n_{so} \cdot t_{so} \cdot t_d.eff\left(\frac{t_d.eff}{2}\right) + A_b\left(h_b - \frac{h_b}{2}\right)}{A_{eff.f.tot}} = 328.891\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.027\,m^3$

Results

The plastic re the flanges effective area:

$M_{pl.Rd} := M_{pl.Rd} = 1.209 \times 10^4\,kN \cdot m$

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.209 \times 10^4\,kN \cdot m$ (Not correct, check stiffeners) $q_{.M} = 85\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.997$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_{.V} = 15\frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.978$

The Reduced stress method

-Different side plate thickness to 1 mm

Material

$f_y(t) := \begin{cases} 355\,MPa & \text{if } t < 40\,mm \\ 345\,MPa & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} := 1$

$E := 210\,GPa$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate:

$t_b := 16mm$ $b_b := 1000mm$ $A_b := b_b t_b = 0.016m^2$

Top plate:

$t_d := 10mm$ $b_d := 3500mm$ $A_d := b_d t_d = 0.035m^2$

Side plates:

$t_s := 1mm$ $h_s := 389.2mm$ $A_s := h_s t_s = 3.892 \times 10^{-4}\,m^2$

Webs:

$t_w := 12mm$ $h_w := 1300mm$ $A_w := h_w t_w = 0.016m^2$

$h_{w,p} := 1250mm$ $h_{w,fl} := 482.8mm$

Top stiffeners:

$t_{st} := 10mm$ $b_{st} := 140mm$ $A_{st} := h_{st} t_{st} = 2.24 \times 10^{-3}\,m^2$

$s_{st} := 350mm$ $n_{st} := 9$

Web stiffeners:

$t_{sw} := 12mm$ $h_{sw} := 120mm$ $A_{sw} := h_{sw} t_{sw} = 1.44 \times 10^{-3}\,m^2$

$s_{sw} := 440mm$ $n_{sw} := 3$

Whole bridge:

$L_{bridge} := 18m$

$h_b := \frac{h_w}{2} + h_{w,p} + t_d + t_b = 0.89m$ $s_b := 2m$

$I_{yd} := 0.014463m^4$ From grasshopper

$V_{fg} := 0.144857m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_{d,1} := 2\,\frac{kN}{m}$

$Q := q_k s_b = 14\,\frac{kN}{m}$

$M_{Ed} := \frac{Q\,L_{bridge}^2}{8} = 567\,kN\,m$

$V_{Ed} := \frac{Q\,L_{bridge}}{2} = 126\,kN$

Top plate:

$\sigma_{x,Ed,tp} := \frac{M_{Ed}}{I_{yd}}\left(h_b - \frac{t_d}{2} - V_{fg}\right) = 14.511\,MPa$ $\sigma_{z,Ed,tp} := 0$

$S_{yd} := \frac{t_d b_d}{n_{st}+1}\left(h_b - \frac{t_d}{2} - V_{fg}\right) = 1.178 \times 10^{-3}\,m^3$

$\tau_{Ed,tp} := \frac{V_{Ed} S_{yd}}{I_{yd} \cdot 4mm\,(t_d + t_{st})} = 10.26\,MPa$

Side plates:

$\sigma_{x,Ed,sp} := \frac{M_{Ed}}{I_{yd}}\left(h_b - t_d - V_{fg}\right) = 14.315\,MPa$

$\sigma_{z,Ed,sp} := \frac{Q_k}{s_b} = 7\,MPa$

$\sigma_{x,Ed,sp,w} := \frac{M_{Ed}}{I_{yd}}\left(h_b - t_d - V_{fg}\right) = 14.315\,MPa$

$S_{sp,w} := \left(\frac{t_{sw} h_{sw}}{2} \cdot 2 + \frac{t_w h_w}{2}\right)\left(h_b - t_d - \frac{h_w}{4} - V_{fg}\right) = 1.484 \times 10^{-3}\,m^3 - 3\,m^3$

$+ \frac{t_d b_d}{n_{st}+1}\left(h_b - \frac{t_d}{2} - V_{fg}\right)$

$\tau_{Ed,sp,w} := \frac{V_{Ed} S_{sp,w}}{I_{yd} \cdot 4m} = 8.808\,MPa$

Top plate stiffeners

Examining slenderness from FE-software

$\sigma_{y,1,UNIT} := 20\,\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

$\sigma_{cp} := \sigma_{y,1,UNIT} q_k = 112\,MPa$

$\alpha_{cr,lb} := \frac{f_y(t)}{\sigma_{cp}} = 3.17$

$\alpha_{cr,1,UNIT} := 6.710\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr,1,UNIT}}{q_k} = 6.18$

$\lambda_p := \sqrt{\frac{\alpha_{cr}}{\alpha_{cr}}} = 4.199$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{p,tp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\,(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.226$

Plate-like behaviour [A.1]

$\sigma_{E,tp} := \frac{\pi^2 E t_d^2}{12\left(1-\nu^2\right) b_d^2} = 1.549\,MPa$

$I_{yp,tp} := \frac{b_d t_d^3}{12\left(1-\nu^2\right)} = 3.205 \times 10^{-7}\,m^4$

$y_{tp,el,tp} := \frac{\frac{t_d}{2} A_d + h_w t_w\left(t_d + \frac{h_w}{2}\right)}{A_d + A_w + A_{st}} = 32.411\,mm$

$I_{y,tp} := \frac{b_d t_d^3}{12} + A_d\left(y_{tp,el,tp} - \frac{t_d}{2}\right)^2 + \frac{t_w h_w^3}{12} + A_w\left(t_d + \frac{h_w}{2} - y_{tp,el,tp}\right)^2 = 1.052 \times 10^{-3}\,mm^4$

$+ A_{st}\left(t_d + h_w + t_{st} - y_{tp,el,tp}\right)^2$

$\gamma_{tp} := \frac{I_{y,tp}}{y_{tp}} = 328.143$

$k_{\sigma,tp} := \frac{A_{st}}{A_d} = 0.576$

$\alpha_{tp} := \min\left(\sqrt{\frac{2m}{k_{\sigma,tp}}}, 6.1\right) = 6.571$

$k_{\sigma,tp,tp} := \begin{cases} \frac{\left(1+\alpha_{tp}^2\cdot\frac{\gamma_{tp}}{\gamma_{tp}}\right)^2 \cdot \left(3+\frac{\gamma_{tp}}{\gamma_{tp}}\right)}{\alpha_{tp}^2\cdot\left(\alpha_{tp}+1\right)\left(1+k_{\sigma,tp}\right)} & \text{if } \alpha_{tp} < \sqrt{\frac{\gamma_{tp}}{\gamma_{tp}}} \\ \frac{4\left(1+\sqrt{\frac{\gamma_{tp}}{\gamma_{tp}}}\right)}{\left(\alpha_{tp}+1\right)\left(1+k_{\sigma,tp}\right)} & \text{if } \alpha_{tp} > \sqrt{\frac{\gamma_{tp}}{\gamma_{tp}}} \end{cases} = 639.127$

$\sigma_{cr,tp,tp} := k_{\sigma,tp,tp}\,\sigma_{E,tp} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1,eff,tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}} b_{sw} = 0.175m$

$b_{2,eff,tp} := \frac{2}{5-\psi_{tp}} b_{sw} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{d,1,tp} := b_{1,eff,tp} + b_{2,eff,tp} = 0.35m$

$y_{tp,el,1,tp} := \frac{\frac{t_d}{2} b_{d,1,tp} t_d + A_{sw}\left(t_d + \frac{h_{sw}}{2}\right)}{b_{d,1,tp} t_d + A_{sw}} = 0.834m$ combined center of gravity for stiffener and contributing top plate.

$I_{d,1,tp} := \frac{b_{d,1,tp} t_d^3}{12} + b_{d,1,tp} t_d\left(y_{tp,el,1,tp} - \frac{t_d}{2}\right)^2 + \frac{t_{sw} h_{sw}^3}{12} + A_{sw}\left(t_d + \frac{h_{sw}}{2} - y_{tp,el,1,tp}\right)^2$

Combined moment of inertia

$\sigma_{cr,el,tp} := \frac{\pi^2 E I_{d,1,tp}}{A_{d,1,tp} b_b^2} = 1.026 \times 10^3\,MPa$ Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,tp} = \sigma_{cr,el}$

$\sigma_{cr,tp} = \sigma_{cr,el,tp}$

Reduction factor for column-like behaviour

$\lambda_{tp} := \sqrt{\frac{f_y}{\sigma_{cr,tp}}} = 0.645m$

$e_{tp} := \max\left(0.15\,y_{tp,el,1,tp} - t_d\,s_b + \frac{h_{sw}}{50}, 0.1\,y_{tp,el,1,tp}\right) = 0.046m$

$\alpha_{e,tp} := 0.49 + \frac{0.09}{\frac{I_{yd}}{A_d}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e,tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 10.402$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1\right) = 0.05$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr,tp}}{\sigma_{cr,el,tp}} - 1.0, 0\right), 1\right) = 0$

$\rho_{c,tp} := \begin{cases} \left[\left(\rho_{p,tp} - \chi_{tp}\right)\xi_{tp}\left(2 - \xi_{tp}\right) + \chi_{tp}\right] \cdot \frac{\gamma_{tp}}{\gamma_{tp}} & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right)\xi_{tp}\left(2 - \xi_{tp}\right) + \chi_{tp}\right]\cdot\gamma_{tp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right)\xi_{tp}\left(2 - \xi_{tp}\right) + \chi_{tp}\right]\cdot\gamma_{tp}}{\gamma_{M1}} \geq 1 \end{cases} = 0.05$

Shear

$\eta := 1.2$ 7.1 (2)

$\lambda_w := \begin{cases} 0 & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \leq \lambda_p < 1.08 \\ \frac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p > 1.08 \end{cases} = 0.28$

$\frac{\sigma_{cr,tp}\,\gamma_{tp}}{\gamma_{M1}} = 0.158$

Side plates check

x-direction:

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(V_{fg}\right)} z + \sigma_{max}\right] - 1$

$\psi_{sp,x} := \frac{\sigma\left(h_b - t_d - h_s\right)}{\sigma\left(h_b - t_d\right)} = -0.066$

$\sigma_{x,sp} := \frac{M_{Ed}}{I_{yd}}\left(h_b - t_d - V_{fg}\right) = 14.315\,MPa$

$\sigma_{E,sp,x} := \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) h_s^2} = 1.253\,MPa$ $0.643\,MPa$

$\psi_{sp3,x} := \max\left(3 \cdot \psi_{sp,x}, 4\right) = 2.934$ $\psi_{sp1} := \frac{\sigma_{x,Ed}}{\sigma_{x,sp}} = -0.066$

$\rho_{p,sp,x} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{\lambda_p - 0.055\,(3+\psi_{sp3,x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.229$

$k_{\sigma,sp,x} := \begin{cases} 8 & \text{if } \psi_{sp,x} \geq 1 \\ \frac{8.2}{1.05 + \psi_{sp,x}^2} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\,\psi_{sp,x} + 9.78\,\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98\left(1 - \psi_{sp,x}\right)^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 35.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 8.267$

$\sigma_{cr,sp,x} := \sigma_{E,sp,x} k_{\sigma,sp,x} = 10.359\,MPa$

Column-like behaviour

$\sigma_{cr,sp,x} := \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) h_w^2} = 0.047\,MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 9.737$

$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1\right) = 0.054$

Interpolation

$\xi_{sp,x} := \min\left(\max\left(\frac{\sigma_{cr,sp,x}}{\sigma_{cr,sp,x}} - 1.0, 0\right), 1\right) = 1$

$\rho_{c,sp,x} := \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2 - \xi_{sp,x}\right) + \chi_{sp}\right] \cdot \frac{\gamma_{tp}}{\gamma_{sp}} & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2 - \xi_{sp,x}\right) + \chi_{sp}\right]\cdot\gamma_{tp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\xi_{sp,x}\left(2 - \xi_{sp,x}\right) + \chi_{sp}\right]\cdot\gamma_{tp}}{\gamma_{M1}} \geq 1 \end{cases} = 0.229$

Z-direction

$\psi_{sp,z} := 1$

$\alpha_{p,sp} := 0.34$ $\lambda_{p0,sp} := 0.7$

$\phi_{p,sp} := 0.5\left[1 + \alpha_{p,sp}\left(\lambda_p - \lambda_{p0,sp}\right) + \lambda_p^2\right] = 1.194$

$\rho_{p1,sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p,sp} + \sqrt{\phi_{p,sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.577$

Plate-like behaviour

$k_{\sigma,p,sp,z} := 4$

$\sigma_{E,sp,z} := \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) h_{sw}^2} = 0.047\,MPa$

$\sigma_{cr,p,sp,z} := k_{\sigma,p,sp,z}\,\sigma_{E,sp,z} = 0.19\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,z} := \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) h_{sw}^2} = 1.253\,MPa$

Interpolation

$\xi_{sp,z} := \min\left(\max\left(\frac{\sigma_{cr,p,sp,z}}{\sigma_{cr,c,sp,z}} - 1.0, 0\right), 1\right) = 0$

$\rho_{c,sp,z} := \begin{cases} \left[\left(\rho_{p1,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2 - \xi_{sp,z}\right) + \chi_{sp}\right] \cdot \frac{\gamma_{tp}}{\gamma_{sp}} & \text{if } \frac{\left[\left(\rho_{p1,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2 - \xi_{sp,z}\right) + \chi_{sp}\right]\cdot\gamma_{tp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p1,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2 - \xi_{sp,z}\right) + \chi_{sp}\right]\cdot\gamma_{tp}}{\gamma_{M1}} \geq 1 \end{cases} = 0.054$

Top plate stiffeners - check

x-direction

$$\rho_{p,c,sp,x} := \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 0.227$$

$$\psi_{sp,c,x} := \frac{\sigma 1[h_w - (t_f + r_{sp})]}{\sigma 1[h_w - t_f]} = 0.617$$

$$\sigma_{E,sp,c,x} := \frac{\pi^2 \cdot E \cdot {t_{sp}}^2}{12 \cdot (1 - \nu^2) \cdot {b_{sp}}^2} = 2.479 \times 10^3 \cdot MPa$$

$$k_{\sigma,sp,c,x} := \begin{cases} 4 & \text{if } \psi_{sp,c,x} \geq 1 \\ \frac{8.2}{1.05 + \psi_{sp,c,x}} & \text{if } 1 > \psi_{sp,c,x} > 0 \\ 7.81 & \text{if } \psi_{sp,c,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp,c,x} + 9.78 \cdot {\psi_{sp,c,x}}^2 & \text{if } 0 > \psi_{sp,c,x} > -1 \\ 23.9 & \text{if } \psi_{sp,c,x} = -1 \\ 5.98 \cdot (1 - \psi_{sp,c,x})^2 & \text{if } -1 > \psi_{sp,c,x} > -3 \\ 85.48 & \text{if } \psi_{sp,c,x} < -3 \end{cases} = 3.974$$

$$\sigma_{cr,sp,c,x} := k_{\sigma,sp,c,x} \cdot \sigma_{E,sp,c,x} = 9.852 \times 10^3 \cdot MPa$$

Columm-like behaviour

$$\sigma_{cr,c,sp,c,x} := \frac{\pi^2 \cdot E \cdot {r_{sp}}^2}{12 \cdot (1 - \nu^2) \cdot {s_{sp}}^2} = 12.147 \cdot MPa$$

$$\chi_{sp,c} := \chi_{sp}$$

Interpolation

$$\psi_{sp,c,x} := \min\left[\max\left(\frac{\sigma_{cr,c,sp,c,x}}{\sigma_{cr,c,sp,c,x}}, 1.0\right), 1\right] = 1$$

$$\rho_{c,sp,c,x} := \begin{cases} \left[\left(\rho_{sp,c,sp} - \chi_{sp}\right)\left[\psi_{sp,c,x}\cdot\left(2 - \psi_{sp,c,x}\right) - 2\right] + 2 \cdot \chi_{sp}\right] \cdot \frac{\gamma_{M0}}{\gamma_{M1}} & \text{if } \frac{\left[\left(\rho_{sp,c,sp} - \chi_{sp}\right)\left[\psi_{sp,c,x}\cdot\left(2 - \psi_{sp,c,x}\right) - 2\right] + 2 \cdot \chi_{sp}\right] \cdot \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{sp,c,sp} - \chi_{sp}\right)\left[\psi_{sp,c,x}\cdot\left(2 - \psi_{sp,c,x}\right) - 2\right] + 2 \cdot \chi_{sp}\right] \cdot \gamma_{M0}}{\gamma_{M1}} \geq 1 \end{cases} = 0.227$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,c,x} \cdot f_y[1]}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y[1]}{\gamma_{M1}}}\right)^2 = 0.706$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,c,x} \cdot f_y[1]}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sp,c,z} \cdot f_y[1]}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,c,x} \cdot f_y[1]}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sp,c,z} \cdot f_y[1]}{\gamma_{M1}}}\right) + 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y[1]}{\gamma_{M1}}}\right)^2 = 0.623$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,c,x} \cdot f_y[1]}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y[1]}{\gamma_{M1}}}\right)^2 = 0.032$$

EWM - Changed side plate thickness

This chapter covers the EWM when changing the side plate thicknessto 3 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s.sd} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{su} := \frac{b_d}{n_{s.su}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 3mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140mm$ $t_{su} := 16mm$ $a_{su} := 6mm$

Stiffeners under: $h_{sl} := 120mm$ $t_{sl} := 12mm$ $a_{sl} := 5mm$

$h_{sl.y} := h_{sl} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{tt} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864 \cdot mm$

Poissons ratio $\upsilon := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d \cdot b_{\text{uraameter}}$ $A_s := 2 \cdot h_s \cdot t_s$ $A_w := 2 \cdot t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{su} := n_{su} \cdot t_{su} \cdot b_{su}$ $A_{sl} := n_{sl} \cdot t_{sl} \cdot b_{sl}$

$E := 210GPa$

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.114 \cdot m^2$ tab. 3.1

$A_c := A_{tot} + A_d = 0.055 \cdot m^2$ ok

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Change of center of gravity, z_{sp}

(From top to centre of gravity)

$$z_{sp} := \frac{A_d\frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b\left(t_d + h_s + h_{wy}\right) + A_{sl}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sl0}\right) - \frac{h_{sl.y}}{2}\right]}{A_{tot}} = 372.811 \cdot mm$$

Moment of inertia I_y

$s_{sl0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{sl0} = 7.206 \cdot mm$

Stiffeners overlunder:

$I_{y.st.su} := n_{su}\frac{t_{su} \cdot h_{su}^3}{12} + n_{su} \cdot t_{su} \cdot h_{su}\left(t_d + \frac{h_{su}}{2} - z_{sp}\right)^2 = 1.761 \times 10^{-3} \cdot m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$I_{y.st.sl} := I_{y.st.su.1} + 2t_{sl}h_{sl}\left[t_d + h_s + \frac{1}{2}\left(h_{wy} - s_{sl0y}\right) - \frac{h_{sl.y}}{2} - z_{sp}\right]^2 \ldots = 8.679 \times 10^{-4} \cdot m^4$
$+ 2t_{sl}h_{sl}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sl0y}\right) - \frac{h_{sl.y}}{2} - z_{sp}\right]^2 \ldots$
$+ 2t_{sl}h_{sl}\left[t_d + h_s + \left(h_{wy} - s_{sl0y}\right) - \frac{h_{sl.y}}{2} - z_{sp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w h_w\left[h_{tb} - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{sp}\right]^2 = 2.93 \times 10^{-3} \cdot m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d\left(\frac{t_d}{2} - z_{sp}\right)^2 = 4.735 \times 10^{-3} \cdot m^4$

Bottom flange:

$I_{y.b} := \frac{b_b t_b^3}{12} + t_b b_b\left(h_{tb} - z_{sp} - \frac{t_b}{2}\right)^2 = 4.149 \times 10^{-3} \cdot m^4$

Side plate:

$I_{y.s} := 2\frac{t_s h_s^3}{12} + 2t_s h_s\left(t_d + \frac{h_s}{2} - z_{sp}\right)^2 = 9.555 \times 10^{-5} \cdot m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015 \cdot m^4$

Reduced cross section

Functions to investigate the stresses

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{sp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{sp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(b_p, t, \varepsilon) := \frac{\frac{b_p}{t}}{28.4 \cdot \varepsilon(t)\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max(3 + \psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.073$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.319$

$s_{s.so} := s_{so} - t_{su}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(b_{sp.4.1}(\psi_s), h_s, t_s) = 1.947$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 0.471$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{sp} = 0.026 \cdot m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 362.811 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 170.968 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 68.387 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 102.581 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 197.357 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 1.026 \times 10^{-3} \cdot mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right) \cdot t_s = 1.184 \times 10^{-3} \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s)\frac{E \cdot \pi^2 \cdot t_s^2}{12\left(1 - \upsilon^2\right)h_s^2} = 93.816 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$h_{sw} := t_d + h_{su} = 150 \cdot mm$

$h_{sb} := t_d + h_{sl} = 130 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.614$

$k_{\sigma,4.2k}(\psi_{so}) = 0.606$ Higher stress close to support

$Class4_{so} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{so}}{t_{so}} > 21 \cdot \epsilon \cdot (\psi_{so}) \cdot \sqrt{k_{\sigma,4.2k}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p,so} := \lambda_p(k_{\sigma,4.2k}(\psi_{so}), b_{so}, t_{so}) = 0.487$

$\rho_{d,so} := \rho_{4.2}(\lambda_{p,so}, \psi_{so}) = 1$ Internal cross-sectional parts, EC1993-1-5:4.4 (2)

$b_{so,eff} := \rho_{d,so} \cdot b_{so} = 140 \cdot mm$

$A_{so,eff} := n_{so} \cdot t_{so} \cdot b_{so,eff} = 2.016 \times 10^{-4} \cdot m^2$

Deck

$\frac{b_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{so} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} \cdot m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$\Delta t_{so} := t_{so} - t_{so} \qquad$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma,d} := k_{\sigma,4.1}(\psi_d) = 4$

$\lambda_{p,d} := \lambda_p(k_{\sigma,d} \cdot s_{c,so} \cdot t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p,d}, \psi_d) = 0.962$ Internal cross-sectional parts

$s_{so,eff} := \rho_d \cdot s_{c,so} = 321.461 \cdot mm$

Areas:

$\Delta A_{so} := n_{so} \cdot s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (b_{so,eff} + t_d) = 5.31 \times 10^{-4} \cdot m^2$

$A_{c,eff.loc} := n_{so} \cdot s_{so,eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (b_{so,eff} + t_d) = 5.053 \times 10^{-4} \cdot m^2$

More than three longitudinal stiffeners -> $k_{\sigma,p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5} \cdot mm^4$

$y_{tp,sl} := \frac{n_{so} \cdot t_{so} \cdot b_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot b_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_d := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp,sl}\right)^2\right] + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp,sl}\right)^2\right] = 1.052 \times 10^{-8} \cdot mm^4$

$\gamma := \frac{I_d}{I_p} = 328.143 \qquad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 \cdot m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035 \cdot m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$\beta_{A.c} := \frac{A_{c,eff.loc}}{A_c} = 0.952$

$\kappa_{\sigma,n} := \begin{cases} \left[\frac{\beta_{A.c} \cdot \sigma_{cr}(t_d)}{\sigma_{cr,p}} - 1\right]^{\psi} & \text{if } \alpha \le \sqrt{\gamma} \\ \left[\frac{\beta_{A.c} \cdot \sigma_{cr}(t_d)}{\sigma_{cr,p}}\right]^{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 639.127$

$\lambda_{p,n} := \sqrt{\frac{\beta_{A.c} \cdot \sigma_y(t_d)}{\sigma_{cr,p}}} = 0.584$

$\rho := \rho_{4.2}(\lambda_{p,n}, \psi) = 1$ Plate like behaviour orthotropic plate:

Effective area of the stiffened plate(deck):

$A_{c,eff} := \rho_{4.2}(\lambda_{p,n}, \psi) \cdot A_{c,eff.loc} + s_{so,eff} \cdot t_d = 5.375 \times 10^{-4} \cdot mm^2$ 3-1-5 eq A.1

Bucking strength

$N_{Rd} := A_{c,eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^4 \cdot kN$

Column like behaviour

$y_{tp,sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^3}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp,sl.c}\right)^2\right] + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp,sl.c}\right)^2\right] = 1.137 \times 10^{-7} \cdot mm^4$

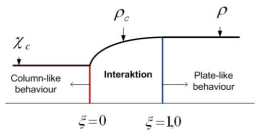
Bucking resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{-3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so,eff} + s_{so,eff} \cdot t_d = 5.455 \times 10^{-3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical bucking stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.026 \times 10^3 \cdot MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sl})}{\sigma_{cr.sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$\Delta e := \max\left[\left(\gamma_{tp,sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - \gamma_{tp,sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{\Delta e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{c.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero -> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{axst} := n_{so} \cdot s_{so,eff} \cdot t_d \cdot \sigma_{neg}\left(t_d + \frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so,eff}}{2}\right) = -1.575 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{axst}}{n_{so} \cdot s_{so,eff} \cdot t_d + A_{so}} = -320.911 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} = 281.032 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EI OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so,eff} = 0.032 \cdot m^2$

$A_{tot.eff} := A_{sl.so.eff} + A_{d.eff} + A_w + A_{fu} + A_{fl} + A_{s.eff} = 0.11 \cdot m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} & A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ & + 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ & \left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{st}}{2}\right)\right] \dots \\ & + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{fu} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{wdy}) - \frac{h_{wy}}{2}\right] \dots \\ & + A_{fl} \cdot \left(h_m - \frac{h_{fl}}{2}\right) \end{aligned}}{A_{tot.eff}} = 384.4 \cdot mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right. \\ \left. + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \right. \\ \left. + \frac{t_s \cdot h_{st}^3}{12} + t_s \cdot h_{st} \cdot \left(t_d + h_s - \frac{h_{st}}{2} - z_{tp.eff.neg}\right)^2\right] = 5.063 \times 10^{-5} \cdot m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.628 \times 10^{-3} \cdot m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s.eff} + I_{y.d.eff} = 0.014 \cdot m^4$

Negative moment

Effective cross-section with negative moment(lower flange in tension)

Area

$A_{td.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^{-4} \cdot m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_{sl}) \cdot A_d}{\sigma_{com.Ed} \cdot \gamma_M} = -5.027 \times 10^{-3} \cdot mm^2$ (A.3)

$A_{td.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

effective moment of inertia

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EI OK"} \\ A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 432.173 \cdot cm^2$

$A_{eff.tot} := A_{c.eff} + A_w + A_{su} + A_{fl} = 1.012 \times 10^{-3} \cdot m^2$

According to SS-EN 1993-1-5:4.5.1 (7) the reduction of the compressive area can be neglected evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

A_b Tension -> No reduction

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EI OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.916 \cdot mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} & n_{so} \cdot h_{so.eff} \cdot s_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ & + n_{so} \cdot s_{so.eff} \cdot t_d \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ & + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{st}}{2}\right) \dots \\ & + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{fu} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{wdy}) - \frac{h_{wy}}{2}\right] \dots \\ & + A_{fl} \cdot \left(h_m - \frac{h_{fl}}{2}\right) \end{aligned}}{A_{eff.tot}} = 413.935 \cdot mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.376 \times 10^{-3} \cdot m^4$

$s_{so.fin} := s_{so}\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{so0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.281 \times 10^{-3} \cdot m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots = 6.782 \times 10^{-4} \cdot m^4$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{2} \cdot (h_{wy} - s_{wdy}) - \frac{h_{wy}}{2} - z_{tp.eff.neg}\right]^2 \dots$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{wdy}) - \frac{h_{wy}}{2} - z_{tp.eff.neg}\right]^2 \dots$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{wdy}) - \frac{h_{wy}}{2} - z_{tp.eff.neg}\right]^2$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 \cdot m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[h_m - \frac{h_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp.eff.neg}\right]^2 = 2.276 \times 10^{-3} \cdot m^4$

Bottom flange:

$I_{y.b.fin} := \frac{h_b \cdot s_b^3}{12} + t_b \cdot h_b \cdot \left(h_m - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.506 \times 10^{-3} \cdot m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right. \\ \left. + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \right. \\ \left. + \frac{t_s \cdot h_{st}^3}{12} + t_s \cdot h_{st} \cdot \left(t_d + h_s - \frac{h_{st}}{2} - z_{tp.eff.neg}\right)^2\right] = 6.218 \times 10^{-5} \cdot m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.418 \times 10^{-10} \cdot mm^4$

Load

Load capacity:

$q_{_M} := 85 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is $q_{_M}$ when checking the moment capacity.

$Q_{_M} := q_{_M} \cdot b_d = 297.5 \cdot \frac{kN}{m}$

$M_{Ed} := \frac{Q_{_M} \cdot l_{bridge}^2}{8} = 1.205 \times 10^4 \cdot kN \cdot m$

$q_{_V} := 25 \frac{kN}{m^2}$ The load capacity is $q_{_V}$ when checking the shear capacity.

$Q_{_V} := q_{_V} \cdot b_d = 87.5 \cdot \frac{kN}{m}$

$V_{Ed} := \frac{Q \cdot V \cdot b_{bridge}}{2} = 787.5\,kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.034\,m^3$

$M_{Rd} := W_{y.eff} \frac{f_y(t_fl)}{\gamma_{M1}} = 1.216 \times 10^4\,kN \cdot m$

Moment capacity

$\frac{M_{Ed}}{M_{Rd}} = 0.991$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeners

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot t(t_w) \cdot t_w = 0.293\,m$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wx}}{h_w}\right) \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) + \frac{t_{ws}}{2} - \frac{\sin\left(\alpha_w \frac{\pi}{180}\right) \cdot h_{ws}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{ss} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{ss} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 1.324\,m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 \cdot t_{ss} \cdot h_{ss} \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$\bar{z}_{web.1} := \frac{h_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) + \frac{t_{ws}}{2} = 0.525\,m$

$\bar{z}_{web.2} := a_{st.z.1} + s_{ss} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 0.935\,m$

$\bar{z}_{web.3} := a_{st.z.2} + s_{ss} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 1.346\,m$

Moment of inirtia of 15st of the web z-direction

$I_{sw.z} := 2 h_{web} \cdot t_w \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{sw.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.379$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_w \cdot \varepsilon(t_s) \sqrt{k_{\tau 1}}} = 0.932$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\frac{a}{h_w} = 1.493$

$k_{\tau sl} := \max\left(5 \left(\frac{h_w}{a}\right)^2 \sqrt[4]{\frac{I_{st.z.tot}}{\left[\min\left(t_s, t_w\right)^3 h_w\right]}}, \frac{2.1}{\min\left(t_s, t_w\right)} \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right) = 1.002 \times 10^5$ EN 1993-1-5 (A.5)

Check := "no shear buckling" if $\frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \frac{\varepsilon(t_s)}{\eta} \sqrt{k_{\tau 1}}$ = "Reduce"

"Reduce" otherwise

$= 1.007 \times 10^2$

Reduce --> Shear buckling check according to EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 \cdot t_w \cdot \varepsilon(t_s) \sqrt{k_\tau}} \lambda_{w1}, 0.932\right) = 0.932$ otherwise

EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 0.89$ rck has to be performed for stiffened webs: SS-EN 1993-1-5 5.3 Table 5.1, stiff end stiffener.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ s.S83 bärande konstruktioner

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 394.192\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 787.5\,kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.999$ (Contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4, S83 bärande konstruktioner

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{fl.eff.neg} := z_{tp.eff.neg} - \frac{t_fl}{2} = 408.935\,mm$

$a_{fl.eff.neg} := h_b - \frac{t_b}{2} - z_{tp.eff.neg} = 468.065\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{fl.eff.neg} \left| a_{fl.eff.neg} \right| \cdot f_y(t_fl)}{\gamma_{M0}} + \frac{A_{sf.eff.neg} \left| a_{sf.eff.neg} \right| \cdot f_y(t_b)}{\gamma_{M0}} = 7.325 \times 10^3\,kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tf} := a \left[0.25 + \frac{1.6\, b_{fl} \cdot t_{fl}^2 \cdot f_y(t_{fl})}{t_s \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.991\,m$

$c_{tf} := a \left[0.25 + \frac{1.6\, b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.859\,m$

$V_{bf.Rd.creed} := \frac{b_{fl} \cdot t_{fl}^2 \cdot f_y(t_{fl})}{c_{tf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{tf} \cdot \gamma_{M1} \cdot \gamma_n} = 192.637\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.creed} \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\,kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} = V_{bw.Rd} + V_{bf.Rd} = 394.192\,kN$

Check:

$\frac{V_{Ed}}{\frac{2}{V_{b.Rd}}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.999$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.882 \times 10^{-3}\,m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{ss.eff} \cdot t_d + n_{ss} \cdot s_{ss.eff} \cdot t_{d.eff} + n_{ss} \cdot t_{ss} \cdot t_{d.eff} + A_b = 432.579\,cm^2$

z_{tp} for the flanges effective areas:

$z_{tp.f.eff.neg} := \frac{s_{ss.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{ss} \cdot s_{ss.eff} \cdot t_{d.eff} \left(\frac{t_{d.eff}}{2}\right) + n_{ss} \cdot t_{ss} \cdot t_{d.eff} \left(\frac{t_{d.eff}}{2}\right) + A_b \left(h_w - \frac{h_b}{2}\right)}{A_{eff.f.tot}} = 328.801\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.027\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \frac{f_y(t_fl)}{\gamma_M} = 9.589 \times 10^3\,kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.216 \times 10^4\,kN \cdot m$

Moment capacity check when $q_M = 85\,\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.991$ EC1993-1-5 eq 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 25\,\frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.999$

The Reduced stress method

-Different side plate thickness to 3 mm

Material

$f_y(t) := \begin{cases} 355MPa & \text{if } t < 40mm \\ 335MPa & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} = 1$

$E := 210GPa$

$\nu := 0.3$ $G := \frac{E}{2 \cdot (1+\nu)} = 80.769 \cdot GPa$

Geometry

Bottom plate

$t_{bp} := 16mm$ $b_{bp} := 1000mm$ $A_{bp} := b_{bp} t_{bp} = 0.016 m^2$

Top plate

$t_{tp} := 16mm$ $b_{tp} := 350mm$ $A_{tp} := b_{tp} t_{tp} = 0.035 m^2$

Side plates

$t_s := 3mm$ $h_s := 389.2mm$ $A_s := h_s t_s = 1.168 \times 10^{-3} m^2$

Webs

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w t_w = 0.016 m^2$

$h_{w.n} := 1250mm$ $h_{w.p} := 492.8mm$

Top stiffeners

$t_{st} := 16mm$ $b_{st} := 140mm$ $A_{st} := b_{st} t_{st} = 2.24 \times 10^{-3} m^2$

$s_{st} := 350mm$ $n_{st} := 9$

Web stiffeners

$t_{wst} := 12mm$ $b_{wst} := 120mm$ $A_{wst} := b_{wst} t_{wst} = 1.44 \times 10^{-3} m^2$

$s_{wst} := 440mm$ $n_{wst} := 3$

Whole bridge

$L_{bridge} := 18m$ $y_{cs} := \frac{h_s}{2} + h_{w.p} + t_{tp} + t_{st} = 0.89 m$ $\gamma_{cs} := 2m$

$I_{yy} := 0.014536m^4$ From grasshopper

$y_{cg} := 0.747634m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 30 \frac{kN}{m^2}$

$Q := 6 b_{tp} = 181 \frac{kN}{m}$

$M_{Ed} := \frac{Q L_{bridge}^2}{8} = 4.253 \times 10^3 kN \cdot m$

$V_{Ed} := \frac{Q L_{bridge}}{2} = 945 kN$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{yy}} \left(h_w - y_{cg} - \frac{t_{tp}}{2} - \gamma_{cs} \right) = 107.796 MPa$ $\sigma_{z.Ed.tp} = 0$

$S_{tp} := \frac{t_{tp} b_{tp}}{n_{st} + 2} \left(h_w - y_{cg} - \frac{t_{tp}}{2} - \gamma_{cs} \right) = 1.171 \times 10^{-3} m^3$

$\tau_{Ed.tp} := \frac{V_{Ed} S_{tp}}{4 I_{yy} (t_s + t_{st})} = 25.406 MPa$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{yy}} \left(h_w - t_{tp} - \gamma_{cs} \right) = 106.331 MPa$

$\sigma_{z.Ed.sp} := \frac{Q}{n_s} = 17.5 MPa$

$S_{sp} := \left(h_w - t_{tp} - \gamma_{cs} \right) t_s \left(\frac{h_w - t_{tp} - \gamma_{cs}}{2} \right) + \frac{t_{tp} b_{tp}}{2} \left(h_w - t_{tp} - \gamma_{cs} \right) = 9.59 \times 10^{-3} m^3$

$+ \frac{b_{tp} t_{tp}}{2} \left(h_w - t_{tp} - \frac{b_{tp}}{2} - \gamma_{cs} \right)$

$\tau_{Ed.sp.} := \frac{V_{Ed} S_{sp}}{4 I_{yy} t_s} = 6.037 MPa$

Governing stresses from FE-software

$\sigma_{x.LIN27} := 10.1 \frac{MPa}{10.4490}$ From a linear stress analysis in Sofistik

$\sigma_{xy} := \sigma_{x.LIN27} \cdot 0.96 = 310 MPa$ $\gamma_{M1} = 106.331 MPa$

$\sigma_{crit} := \frac{\pi^2 E}{\left(\frac{s_{st}}{t_s} \right)^2} \cdot 1.116 \cdot \left(\frac{t_s}{s_{st}} \right)^2 \left(h_w - t_{tp} - \frac{h_{st}}{2} - \gamma_{cs} \right) + \frac{t_{tp} b_{tp}}{n_{st} + 2} \left(h_w - t_{tp} - \frac{t_{tp}}{2} - \gamma_{cs} \right) = 1.484 \times 10^{-3} m^3$

$\sigma_{x.LIN27} := 11.876 \frac{MPa}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\sigma_{x.LIN27}}{\sigma_{xy}} = 0.533$

$\lambda_p := \sqrt{\frac{\sigma_{xy}}{\sigma_{cr}}} = 1.448$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{tp.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055 \cdot (3 + \psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.586$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2 E t_{tp}^2}{12 \left(1 - \nu^2 \right) b_{tp}^2} = 1.549 MPa$

$I_{y.tp} := \frac{b_{tp} t_{tp}^3}{12 \left(1 - \nu^2 \right)} = 3.205 \times 10^{-7} m^4$

$y_{tp.el.tp} := \frac{A_{tp} \frac{t_{tp}}{2} + n_{st} A_{st} \left(t_{tp} + \frac{h_{st}}{2} \right)}{A_{tp} + n_{st} A_{st}} = 32.411 mm$

$I_{y.tp} := \frac{b_{tp} t_{tp}^3}{12} + A_{tp} \left(y_{tp.el.tp} - \frac{t_{tp}}{2} \right)^2 + n_{st} \left[\frac{t_{st} h_{st}^3}{12} + A_{st} \left(t_{tp} + \frac{h_{st}}{2} - y_{tp.el.tp} \right)^2 \right] = 1.652 \times 10^{-6} mm^4$

$n_{st} = 9$

$\gamma_{tp} := \frac{I_{y.tp}}{y_{tp.tp}} = 328.143$

$k_{tp} := \frac{n_{st} A_{st}}{A_{tp}} = 0.576$

$\alpha_{tp} := \min \left(\frac{\gamma_{tp}}{\sqrt{k_{tp}}}, 0.5 \right) = 0.571$ $\sqrt{k_{tp}} = 4.256$

$k_{\sigma.tp.tp} := \begin{cases} \frac{2 \left[\left(1 + \alpha_{tp} \right)^2 + \gamma_{tp} - 1 \right]}{\alpha_{tp}^2 \left(6 \gamma_{tp} + 1 \right) \left(1 + \delta_{tp} \right)} & \text{if } \alpha_{tp} \cdot \sqrt{\gamma_{tp}} < 0.639.127 \\ \frac{4 \left(1 + \sqrt{\gamma_{tp}} \right)}{\left(6 \gamma_{tp} + 1 \right) \left(1 + \delta_{tp} \right)} & \text{if } \alpha_{tp} \cdot \sqrt{\gamma_{tp}} \end{cases}$

$\sigma_{cr.tp.tp} := k_{\sigma.tp.tp} \sigma_{E.tp.tp} = 990.256 MPa$

Column-like behaviour [A.5.3]

$b_{1.ad.tp} := \frac{1 - \psi_{tp}}{3 - \psi_{tp}} b_{tp} = 0.175 m$

$b_{2.sup.tp} := \frac{2}{3 - \psi_{tp}} b_{tp} = 0.175 m$ Parts of the top plate that contributes to the stiffener

$b_{tp.1.tp} := b_{1.ad.tp} + b_{2.sup.tp} = 0.35 m$

$y_{tp.el.1.tp} := \frac{t_{tp} b_{tp.1.tp} \frac{t_{tp}}{2} + A_{st} \left(t_{tp} + \frac{h_{st}}{2} \right)}{t_{tp} b_{tp.1.tp} + A_{st}} = 0.114 m$ combined center of gravity for stiffener and contributing top plate

$I_{tp.1.tp} := \frac{b_{tp.1.tp} t_{tp}^3}{12} + b_{tp.1.tp} t_{tp} \left(y_{tp.el.1.tp} - \frac{t_{tp}}{2} \right)^2 + \frac{t_{st} h_{st}^3}{12} + A_{st} \left(t_{tp} + \frac{h_{st}}{2} - y_{tp.el.1.tp} \right)^2$

Combined moment of inertia

$\sigma_{cr.el.tp} := \frac{\pi^2 E I_{tp.1.tp}}{A_{tp.1.tp} \gamma_{cs}^2} = 1.626 \times 10^3 MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.p} = \sigma_{cr.el}$

$\sigma_{cr.c.tp} := \sigma_{cr.el.tp}$

Reduction factor for column-like behaviour

$\lambda_p := \sqrt{\frac{\gamma_{tp} f_y}{\sigma_{cr.el.tp}}} = 0.045 m$

$e_{tp} := \max \left(y_{tp.el.1.tp} - t_{tp} - t_{st} + \frac{h_{st}}{2} - y_{tp.el.1.tp}, 0 \right) = 0.846 m$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{h_{st}}{t_{st}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5 \left[1 + \alpha_{e.tp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 1.012$ [6.3.1.2]

$\chi_{tp} := \min \left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1 \right) = 0.596$

Interpolation

$t_{tp} := \min \left(\max \left(\frac{\sigma_{cr.c.tp}}{\sigma_{cr.el.tp}} - 1.0 \right), 1 \right) = 0$

$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{tp.tp} - \chi_{tp} \right) \chi_{tp} \left(2 - \chi_{tp} \right) - \right] & \text{if } \frac{\left[\left(\rho_{tp.tp} - \chi_{tp} \right) \chi_{tp} \left(2 - \chi_{tp} \right) - \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{tp.tp} - \chi_{tp} \right) \chi_{tp} \left(2 - \chi_{tp} \right) + \chi_{tp}^2 \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.596$

Shear

$\eta_3 := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta_3 & \text{if } \lambda_p < \frac{0.83}{\eta_3} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_3} \le \lambda_p < 1.08 \\ \frac{0.37}{\left(0.7 + \lambda_p \right)} & \text{if } \lambda_p > 1.08 \end{cases} = 0.638$

$\frac{\sigma_{x.sp} \gamma_{M0}}{\gamma_{M1}} = 0.353$

Side plates check

x-direction

$\sigma_{max} := f_y(t_s)$

$\sigma_{E2} := \left[\frac{-\sigma_{max}}{1 \cdot \tau_{Ed}} + \tau_{max} \right] - 1$

$\psi_{sp.x} := \frac{\sigma \left(h_w - t_{tp} - h_s \right)}{\sigma \left(h_w - t_{tp} \right)} = -0.072$

$\sigma_{x.top} := \frac{M_{Ed}}{I_{yy}} \left(h_w - t_{tp} - \gamma_{cs} \right) = 106.331 MPa$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{yy}} \left(h_w - t_{tp} - h_s - \gamma_{cs} \right) = -7.686 MPa$

$\psi_{sp3.x} := \max \left(1 \cdot \psi_{sp.x}, 0 \right) = 2.928$ $\psi_{sp3} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.072$

$\rho_{p.sp.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055 \left(3 + \psi_{sp3} \right)}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.614$

Plate-like behaviour

$\sigma_{E.sp.x} := \frac{\pi^2 E t_s^2}{12 \left(1 - \nu^2 \right) b_{sp}^2} = 11.277 MPa$

$\sigma_{cr.sp.sp.x} := \sigma_{E.sp.x} k_{\sigma.sp.x} = 93.777 MPa$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} \ge 1 \\ 7.81 - 6 \psi_{sp.x} & \text{if } 1 > \psi_{sp.x} > 0 \\ \frac{7.81}{1.81 - \psi_{sp.x}} & \text{if } 0 > \psi_{sp.x} > -1 \end{cases} = 8.316$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2 E t_s^2}{12 \left(1 - \nu^2 \right) h_s^2} = 0.427 MPa$

$\alpha_{sp} := \begin{cases} 23.9 & \text{if } \psi_{sp.x} \ge -1 \\ 5.98 \left(1 - \psi_{sp.x} \right)^2 & \text{if } -1 > \psi_{sp.x} > -3 \\ 85.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 0.24$

$\phi_{sp} := 0.5 \left[1 + \alpha_{sp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 1.679$

$\chi_{sp} := \min \left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1 \right) = 0.295$

Interpolation

$t_{sp.x} := \min \left(\max \left(\frac{\sigma_{cr.sp.sp.x}}{\sigma_{cr.c.sp.x}} - 1.0 \right), 1 \right) = 1$

$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{sp.sp} - \chi_{sp} \right) \chi_{sp} \left(2 - \chi_{sp} \right) - \right] & \text{if } \frac{\left[\left(\rho_{sp.sp} - \chi_{sp} \right) \chi_{sp} \left(2 - \chi_{sp} \right) - \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{sp.sp} - \chi_{sp} \right) \chi_{sp} \left(2 - \chi_{sp} \right) + \chi_{sp}^2 \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.614$

Z-direction

$\psi_{sp.z} = 1$

$\alpha_{sp.sp} := 0.34$ $\chi_{sp.sp} := 0.7$

$\phi_{sp.sp} := 0.5 \left[1 + \alpha_{sp.sp} \left(\lambda_p - \chi_{sp.sp} \right) + \lambda_p^2 \right] = 1.251$

$\rho_{sp.sp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{sp.sp} + \sqrt{\phi_{sp.sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.508$

Plate-like behaviour

$k_{\sigma.sp.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2 E t_s^2}{12 \left(1 - \nu^2 \right) b_{sp}^2} = 0.427 MPa$

$\sigma_{cr.sp.sp.z} := k_{\sigma.sp.sp.z} \sigma_{E.sp.z} = 1.708 MPa$

Column-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 E t_s^2}{12 \left(1 - \nu^2 \right) h_s^2} = 11.277 MPa$

Interpolation

$t_{sp.z} := \min \left(\max \left(\frac{\sigma_{cr.sp.sp.z}}{\sigma_{cr.c.sp.z}} - 1.0 \right), 1 \right) = 0$

$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{sp.sp} - \chi_{sp} \right) \chi_{sp} \left(2 - \chi_{sp} \right) - \right] & \text{if } \frac{\left[\left(\rho_{sp.sp} - \chi_{sp} \right) \chi_{sp} \left(2 - \chi_{sp} \right) - \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{sp.sp} - \chi_{sp} \right) \chi_{sp} \left(2 - \chi_{sp} \right) + \chi_{sp}^2 \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.295$

Top plate stiffeners - check

x-direction

$\rho_{p.x.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 0.601$

$\psi_{sp.x} := \frac{\sigma \left(h_w - t_{tp} - h_{st} \right)}{\sigma \left(h_w - t_{tp} \right)} = 0.414$

$\sigma_{E.sp.x} := \frac{\pi^2 E t_{st}^2}{12 \left(1 - \nu^2 \right) b_{sp}^2} = 2.479 \times 10^3 MPa$

$$k_{\sigma,\psi,b,A} := \begin{cases} 5 & \text{if } \psi_{\psi p,b,A} \geq 1 \\ \frac{8.2}{1.05 + \psi_{\psi p,b,A}} & \text{if } 1 > \psi_{\psi p,b,A} > 0 \\ 7.81 & \text{if } \psi_{\psi p,b,A} \geq 0 \\ 7.81 - 6.29\cdot\psi_{\psi p,b,A} + 9.78\cdot\psi_{\psi p,b,A}^2 & \text{if } 0 > \psi_{\psi p,b,A} > -1 \\ 23.9 & \text{if } \psi_{\psi p,b,A} \geq -1 \\ 5.98\cdot(1 - \psi_{\psi p,b,A})^2 & \text{if } -1 > \psi_{\psi p,b,A} > -3 \\ 41.68 & \text{if } \psi_{\psi p,b,A} < -3 \end{cases} = 3.997$$

$$\sigma_{cr,p,b,\psi p,b,A} := k_{\sigma,\psi p,b,A} \cdot \sigma_{E,p,b,A} = 9.989 \times 10^3 \cdot MPa$$

Column-like behaviour

$$\sigma_{cr,c,\psi p,b,A} := \frac{\pi^2 \cdot E \cdot t_{wb}^2}{12 \cdot (1 - \nu^2) \cdot b_{wb}^2} = 12.147 \cdot MPa$$

$$\chi_{\psi p,b} := \chi_{\psi p}$$

Interpolation

$$\xi_{\psi p,b,A} := \min\left(\max\left(\frac{\sigma_{cr,\psi p,b,A}}{\sigma_{cr,c,\psi p,b,A}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c,\psi p,b,A} := \begin{cases} \left[\left(\rho_{ax,\psi p} - \chi_{\psi p}\right)\cdot\left(\xi_{\psi p,b,A}\cdot\left(2 - \xi_{\psi p,b,A}\right)\right)^{n_{\psi p}}\right] + \chi_{\psi p} & \text{if } \frac{\left[\left(\rho_{ax,\psi p} - \chi_{\psi p}\right)\cdot\left(\xi_{\psi p,b,A}\cdot\left(2 - \xi_{\psi p,b,A}\right)\right)^{n_{\psi p}}\right] + \chi_{\psi p}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{ax,\psi p} - \chi_{\psi p}\right)\cdot\left(\xi_{\psi p,b,A}\cdot\left(2 - \xi_{\psi p,b,A}\right)\right) + \chi_{\psi p}\right]^{n_{\psi p}}}{\gamma_{M1}} \geq 1 \end{cases} = 0.601$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,wp,A}\cdot f_y b_{sp}}{\gamma_{M1}}}\right)^2 + 1\cdot\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w\cdot f_y b_{sp}}{\gamma_{M1}}}\right)^2 = 0.958$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,wp,A}\cdot f_y b_{sp}}{\gamma_{M1}}}\right)^2 + 1\cdot\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,wp,A}\cdot f_y b_{sp}}{\gamma_{M1}}}\right)^2 - 1\cdot\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,wp,A}\cdot f_y b_{sp}}{\gamma_{M1}}}\right)\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,wp,A}\cdot f_y b_{sp}}{\gamma_{M1}}}\right) + 3\cdot\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w\cdot f_y b_{sp}}{\gamma_{M1}}}\right)^2 = 0.826$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,wp,A}\cdot f_y b_{sp}}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w\cdot f_y b_{sp}}{\gamma_{M1}}}\right)^2 = 0.251$$

EWM - Changed side plate thickness

This chapter covers the EWM when changing the side plate thickness to 4 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s.sd} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{sd} := \frac{b_d}{n_{s.sd}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 4mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{sd0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140mm$ $t_{su} := 16mm$ $a_{su} := 6mm$

Stiffeners under: $h_{sl} := 120mm$ $t_{sl} := 12mm$ $a_{sl} := 5mm$

$h_{sl.y} := h_{sl} \cdot \cos\left(21.12 \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{tb} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{tb} := 2000mm$ $s_{tb0} := 500mm$

$t_{tb.1} := 18mm$ $t_{tb2} := 12mm$ $t_{tb3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

$A_d := t_d \cdot b_d$ $A_s := 2 \cdot t_s \cdot h_s$ $A_w := 2 \cdot t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$ $A_{sl} := n_{sl} \cdot t_{sl} \cdot h_{sl}$

Material parameters:

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.115 \cdot m^2$

$A_c := A_{su} + A_d = 0.055 \cdot m^2$ 3mm EC1993-1-1 tab. 3.1

$f_y := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 335MPa & \text{otherwise} \end{cases}$

Change of center of gravity, z_{tp}

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

$A_d \cdot \left(t_d + h_s + h_{wy} + \frac{t_d}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_{su} \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots$

$+ A_b \cdot \left(t_d + h_s + h_{wy}\right) + A_{sl} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sd0}\right) - \frac{h_{wy}}{2}\right]$

$z_{tp} := \frac{}{A_{tot}} = 371.673 \cdot mm$

Moment of inertia I_y

$s_{sd0y} := \sin\left(21.12 \frac{\pi}{180}\right) \cdot s_{sd0} = 7.206 \cdot mm$

Stiffeners overlunder:

$I_{y.st.su} := n_{su} \cdot \left[\frac{t_{su} \cdot h_{su}^3}{12} + t_{su} \cdot h_{su} \cdot \left(t_d + \frac{h_{su}}{2} - z_{tp}\right)^2\right] = 1.748 \times 10^{-7} \cdot m^4$

$I_{y.st.sl.1} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$I_{y.st.sl} := I_{y.st.sl.1} + 2 \cdot t_{sl} \cdot h_{sl} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{sd0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2 \ldots = 8.735 \times 10^{-4} \cdot m^4$

$+ 2 \cdot t_{sl} \cdot h_{sl} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sd0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2 \ldots$

$+ 2 \cdot t_{sl} \cdot h_{sl} \cdot \left[t_d + h_s + \left(h_{wy} - s_{sd0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 \cdot t_w \cdot h_w \cdot \left[h_b - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 2.95 \times 10^{-3} \cdot m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.706 \times 10^{-3} \cdot m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{tb} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.167 \times 10^{-4} \cdot m^4$

Side plate:

$I_{y.s} := 2 \cdot \frac{t_s \cdot h_s^3}{12} + 2 \cdot t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 1.262 \times 10^{-4} \cdot m^4$

Total:

$I_{y.tot} := I_{y.st.su} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015 \cdot m^4$

Reduced cross section

Functions to investigate the stresses

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \ge -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(b_w, h, \varepsilon) := \frac{\frac{b_w}{\tau}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left(\frac{\lambda_p - 0.055 \cdot \max(3 + \psi), 0]}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.076$

$\text{Condition} := \begin{cases} 62 \cdot \varepsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \le -1 \\ 42 \cdot \varepsilon(t_s) & \text{otherwise} \\ 0.67 + 0.33 \cdot \psi_s & \end{cases}$

$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$k_{\sigma.4.1}(\psi_s) = 8.345$

$s_{c.s0} := s_{s0} - t_{s0}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(b_{w.4.1}(\psi_s), h_s, t_s) = 1.458$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 0.61$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.028 \cdot m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 361.673 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 220.748 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 88.299 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 132.449 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 248.275 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 1.766 \times 10^{-3} \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 1.986 \times 10^{-3} \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \frac{E \cdot \pi^2 \cdot t_s^2}{12\left(1 - \nu^2\right) b_s^2} = 167.308 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$t_w := t_d = 10 \cdot mm$

$h_{sw} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.613$

$k_{\sigma.4.2}(\psi_{so}) = 0.607$ Higher stress close to support

$Class4_{sw} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21\cdot\varepsilon\cdot\sqrt{k_{\sigma.4.2}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2}(\psi_{so}), b_{so}, t_{so}) = 0.486$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so}\cdot b_{so} = 140\ mm$

$A_{so.eff} := n_{so}\cdot t_{so}\cdot b_{so.eff} = 2.016 \times 10^{-4}\ m^2$

Deck

$\frac{\lambda_{so}}{t_d} = 35 \qquad 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{so} := \begin{cases} \text{"Class4"} & \text{if } \frac{\lambda_{so}}{t_d} > 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5\times10^{-3}\ m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$\lambda_{sw} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}\cdot s_{cav}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d\cdot s_{cav} = 321.461\ mm$

Areas:

$A_{sw} := n_{so}\cdot s_{so}\cdot t_d + n_{so}\cdot t_{so}\cdot(h_{so.eff} + t_d) = 5.31\times10^{-4}\ m^2$

$A_{c.eff.loc} := n_{so}\cdot s_{so.eff}\cdot t_d + n_{so}\cdot t_{so}\cdot(h_{so.eff} + t_d) = 5.053\times10^{-4}\ m^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-5}\ m^4$

$y_{tp.sl} := \frac{n_{so}\cdot t_{so}\cdot h_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right) + b_d\cdot t_d\cdot\frac{t_d}{2}}{n_{so}\cdot s_{so}\cdot t_{so} + b_d\cdot t_d} = 32.411\ mm$

$I_d := \left[\frac{b_d\cdot t_d^3}{12} + b_d\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] + \left[\frac{n_{so}\cdot t_{so}\cdot h_{so}^3}{12} + n_{so}\cdot h_{so}\cdot t_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052\times10^{-8}\ m^4$

$\gamma := \frac{\lambda_d}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so}\cdot h_{so}\cdot t_{so} = 0.02\ m^2$ EC1993-1-5A.1 (2)

$A_p := b_d\cdot t_d = 0.035\ m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k := \begin{cases} \frac{2\cdot\left[(1+\alpha^2)^2 + \gamma - 1\right]}{\alpha^2\cdot(\psi+1)\cdot(1+\delta)} & \text{if } \alpha \le \sqrt{\gamma} \\ 4 + \frac{2}{\alpha^2} + \frac{\gamma\cdot(1+\delta)}{\alpha^2} & \text{otherwise} \end{cases} = 639.127$

$\lambda_{cp} := \sqrt{\frac{\sigma_{cr.p}\cdot A_p(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Critical buckling stress, orthotropic plate:

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi)\cdot A_{c.eff.loc} + s_{so.eff}\cdot t_d = 5.375\times10^{-4}\ m^2$ 7.3-1-5 eq A.1

Buckling reduction factor $\chi_{c.z} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$N_{Rd} := A_{c.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 1.908\times10^{4}\ kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so}\cdot h_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right) + s_{so}\cdot t_d\cdot\frac{t_d}{2}}{t_{so}\cdot h_{so} + s_{so}\cdot t_d} = 34.268\ mm$

$I_{sl.c} := \left[\frac{s_{so}\cdot t_d^3}{12} + s_{so}\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] + \left[\frac{t_{so}\cdot h_{so}^3}{12} + h_{so}\cdot t_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137\times10^{-7}\ m^4$

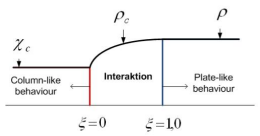
Buckling resistance stiffener

$A_{sl} := t_{so}\cdot h_{so} + s_{so}\cdot t_d = 5.74\times10^{-3}\ m^2$

$A_{sl.eff} := t_{so}\cdot h_{so.eff} + s_{so.eff}\cdot t_d = 5.455\times10^{-3}\ m^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2\cdot E\cdot I_{sl.c}}{A_{sl}\cdot a^2} = 1.026\times10^{3}\ MPa$

Slenderness ratio $\lambda_{c.sl} := \sqrt{\frac{\beta_c\cdot f_y(t_{sw})}{\sigma_{cr.sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\ mm$

$e_{sl} := \min\left(\left[y_{tp.sl} - \frac{t_d}{2}\right], \left[\left(t_d + \frac{h_{so}}{2}\right) - y_{tp.sl}\right]\right) = 47.589\ mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5\cdot\left[1 + \alpha_e\cdot(\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\frac{\sigma_{c.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c)\cdot\xi\cdot(2 - \xi) + \chi_c = 0.792$

Checks

Mean stress

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so}\cdot s_{so.eff}\cdot t_d\cdot\sigma_{neg}\left(z_d + \frac{t_d}{2}\right) + A_{so}\cdot\sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.575\times10^{4}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so}\cdot s_{so.eff}\cdot t_d + A_{so}} = -320.806\cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c\cdot f_y(t_{sw})}{\gamma_M} = 281.032\cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c\cdot f_y(t_{sw})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{so}\cdot t_d\cdot s_{so.eff} = 0.032\ m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{s.eff} = 0.111\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff}\cdot\frac{t_d}{2} + A_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2}\right) + \\ &+ 2\cdot\left[t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2}\right) + \right. \\ &\left. + t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{st.t}}{2}\right)\right] + \\ &+ A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so}\cdot\left[t_d + h_s + \frac{2}{3}\cdot(h_{wy} - s_{sdy}) - \frac{h_{so.t}}{2}\right] + \\ &+ A_b\cdot\left(h_w - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 383.152\ mm$

$I_{y.s.eff} := 2\cdot\left[\frac{t_s\cdot h_{s.eff.c1}^3}{12} + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \right. \\ \left. + \frac{t_s\cdot h_{s.eff.c2}^3}{12} + t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \right. \\ \left. + \frac{t_s\cdot h_{s.t}^3}{12} + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 8.488\times10^{-5}\ m^4$

$I_{y.d.eff} := \frac{n_{so}\cdot s_{so.eff}\cdot t_d^3}{12} + n_{so}\cdot t_d\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.597\times10^{-3}\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\ m^4$

Negative moment

Effective cross-section with negative moment(lower flange in tension)

Area

$A_{tot.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053\times10^{-4}\ m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c\cdot f_y(t_d)\cdot A_{cl}}{\sigma_{com.Ed}\cdot\gamma_M} = -5.028\times10^{-3}\ m^2$ (A.3)

$A_{sl.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_w + A_{so} + A_b = 1.02\times10^{-3}\ m^2$

According to ES-EN 1993-1-5 4.5.1 (7) the reduction of the compressive area can be neglected even if established over the cross section. Therefore the web-thickness is changed with another area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c\cdot\frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot t_d \end{cases} = 7.916\cdot mm$

Effective areas, effective centre of gravity and

$z_{tp.eff.tot} := \frac{\begin{aligned} &n_{so}\cdot h_{so.eff}\cdot t_{so}\cdot\left(t_{d.eff} + \frac{h_{so}}{2}\right) + s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} - \\ &+ n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_d}{2}\right) + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2}\right) - \\ &+ t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{st.t}}{2}\right) - \\ &+ A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so}\cdot\left[t_d + h_s + \frac{2}{3}\cdot(h_{wy} - s_{sdy}) - \frac{h_{so.t}}{2}\right] - \\ &+ A_b\cdot\left(h_w - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 411.512\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so}\cdot s_{so.eff}\cdot t_{d.eff}^3}{12} + t_d\cdot n_{so}\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.312\times10^{-3}\ m^4$

$\alpha_{web} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot rad = 7.206\cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so}\cdot\frac{t_{so}\cdot h_{so.eff}^3}{12} + s_{so}\cdot t_{so}\cdot h_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.249\times10^{-3}\ m^4$

$I_{y.st.su.1.fin} = 6\cdot1.506\cdot10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} - \\ + 2\cdot t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \frac{1}{3}\cdot(h_{wy} - s_{sdy}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 - \\ + 2\cdot t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \frac{2}{3}\cdot(h_{wy} - s_{sdy}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 - \\ + 2\cdot t_{su}\cdot h_{su}\cdot\left[t_d + h_s + (h_{wy} - s_{sdy}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 = 6.885\times10^{-4}\ m^4$

Web:

$I_{y.w.1.fin} := 2\cdot0.000312\ m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w\cdot h_w\cdot\left[h_w - \frac{h_b}{2} - \frac{1}{2}\cdot(h_{wy}) - z_{tp.eff.neg}\right]^2 = 2.312\times10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b\cdot h_b^3}{12} + t_b\cdot b_b\cdot\left(h_w - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 3.542\times10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2\cdot\left[\frac{t_s\cdot h_{s.eff.c1}^3}{12} + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \right. \\ \left. + \frac{t_s\cdot h_{s.eff.c2}^3}{12} + t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \right. \\ \left. + \frac{t_s\cdot h_{s.t}^3}{12} + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 1.043\times10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.421\times10^{-2}\ m^4$

Load

Load capacity:

$q_{_M} := 86\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_{dg} when checking the moment capacity.

$Q_{_M} := q_{_M}\cdot b_d = 301\ \frac{kN}{m}$

$M_{Ed} := \frac{Q_{_M}\cdot l_{bridge}^2}{8} = 1.219\times10^{4}\ kN\cdot m$

$q_{_V} := 35\frac{kN}{m^2}$ The load capacity is q_d when checking the shear capacity.

$Q_{_V} := q_{_V}\cdot b_d = 122.5\ \frac{kN}{m}$

$V_{Ed} := \frac{Q_{-}V_{-}h_{bridge}}{2} = 1.103 \times 10^{3}\,kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{sp.eff.neg}} = 0.035\,m^{3}$

$M_{Rd} := W_{y.eff} \frac{f_y(t_d)}{\gamma_{M1}} = 1.226 \times 10^{4}\,kN \cdot m$

Moment capacity

$\frac{M_{Ed}}{M_{Rd}} = 0.995$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot t_d + t_w = 0.293\,m$

$\alpha_w := \min\left(\frac{h_{web}}{h_w}, \frac{180}{\pi}\right) = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_h}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ws}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{ns}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{ns} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{ns} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 \cdot t_{ns} \cdot h_{ns} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^{4}$

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$a_{web.1} := \frac{h_h}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ws}}{2} = 0.525\,m$

$a_{web.1} := a_{st.z.1} + s_{ns} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$

$a_{web.2} := a_{st.z.2} + s_{ns} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$

Moment of inirtia of 15st of the web z-direction

$I_{sw.z} := 2 \cdot h_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^{4}$

$I_{st.z.tot} := I_{st.z} + I_{sw.z} = 0.029\,m^{4}$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.402$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{wt} = 0$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4 \cdot t_w \cdot \varepsilon(t_d) \cdot \sqrt{k_{\tau 1}}} = 0.878$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_{st} := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau 2} := \max\left[9 \cdot \left(\frac{h_w}{a}\right)^2 \cdot \sqrt{\frac{I_{st.z.tot}}{\left(\max\left(t_1, t_w\right)^3 \cdot h_w\right)}}, \frac{2.1}{\min\left(t_1, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 5.664 \times 10^{4}$

$\gamma_{M1} := 1.2$ EC1993-1-5 ANM.2 5.1

SS-EN 1993-1-5 A.3 (1) and (2) apl

Check := "no shear buckling" if $\frac{h_{web}}{\min\left(t_1, t_w\right)} \leq 31 \cdot \frac{\varepsilon(t_d)}{\eta} \cdot \sqrt{k_{\tau 1}}$; "Reduce" = "Reduce" otherwise

Reduce --> Shear buckling check is needed, EC1993-1-5 Ch. 5.1(2)

$\lambda_w := \max\left(\frac{h_w}{37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.878$ $\quad 2.2 \cdot \sqrt{\frac{I_{st.z.tot}}{\min\left(t_1, t_w\right)^3 \cdot h_w}}$ otherwise SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 0.946$ It has to be performed a check of the web and stiffeners SS-EN 1993-1-5 5.1, stiff and stiffener.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\quad \gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ s.583 bärande konstruktioner

Shear capacity.

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_1) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1} \cdot \eta} = 558.239\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 1.103 \times 10^{3}\,kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.987$ (Contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_1) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4, S83 bärande konstruktioner

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{nf.eff.neg} := z_{np.eff.neg} - \frac{t_d}{2} = 406.512\,mm$

$z_{nf.eff.neg} := h_h - \frac{h_h}{2} - z_{np.eff.neg} = 470.488\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{nf.eff.neg} \cdot \left|z_{nf.eff.neg}\right| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{wf.eff.neg} \cdot \left|z_{wf.eff.neg}\right| \cdot f_y(t_1)}{\gamma_{M0}} = 7.311 \times 10^{3}\,kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{nf} := \min\left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_1 \cdot \left(b_s + b_{wy}\right)^2 \cdot f_y(t_1)}\right] = 0.868\,m$

$c_{wf} := \min\left[0.25 + \frac{1.6 \cdot b_h \cdot t_h^2 \cdot f_y(t_h)}{t_1 \cdot \left(b_s + b_{wy}\right)^2 \cdot f_y(t_1)}\right] = 0.769\,m$

$V_{bf.Rd.creed} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{nf} \cdot \gamma_{M1} \cdot \eta} + \frac{b_h \cdot t_h^2 \cdot f_y(t_h)}{c_{wf} \cdot \gamma_{M1} \cdot \eta} = 217.695\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.creed} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\,kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \cdot \frac{\eta \cdot f_y(t_1) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{stiffw} := V_{bw.Rd} + V_{bf.Rd} = 558.239\,kN$

Check:

$\frac{V_{Ed}}{2} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.987$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.855 \times 10^{-3}\,m^{4}$

The flanges effective area:

$A_{eff.f.tot} := s_{ns.eff} \cdot t_d + n_{ns} \cdot s_{ns.eff} \cdot t_{d.eff} + n_{ws} \cdot t_{ws} \cdot t_{d.eff} + A_h = 432.579\,cm^{2}$

$z_{np.f.eff.neg} := \frac{s_{ns.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{ns} \cdot s_{ns.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + \ldots + n_{ns} \cdot t_{ws} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_h \cdot \left(h_h - \frac{h_h}{2}\right)}{A_{eff.f.tot}} = 328.801\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{np.f.eff.neg}} = 0.027\,m^{3}$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 9.56 \times 10^{3}\,kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.226 \times 10^{4}\,kN \cdot m$

Moment capacity check when $q_M = 86\,\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.995$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 35\,\frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.987$

The Reduced stress method

- Changed side plate thickness to 4 mm

Material

$f_{y0} := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 335MPa & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} = 1$

$E = 210GPa$

$\nu = 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$h_b := 16mm$ $b_b := 1000mm$ $A_b := b_b h_b = 0.016m^2$

Top plate

$t_p := 10mm$ $b_p := 3500mm$ $A_p := b_p t_p = 0.035m^2$

Side plates

$t_s := 4mm$ $h_s := 389.2mm$ $A_s := h_s t_s = 1.557\times10^{-3}m^2$

Webs

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w t_w = 0.016m^2$

$h_{w,h} := 1250mm$ $h_{w,v} := 482.8mm$

Top stiffeners

$t_{st} := 16mm$ $b_{st} := 140mm$ $A_{st} := h_{st} t_{st} = 2.24\times10^{-3}m^2$

$s_{st} := 350mm$ $n_{st} := 9$

Web stiffeners

$t_{wst} := 12mm$ $h_{wst} := 120mm$ $A_{wst} := h_{wst} t_{wst} = 1.44\times10^{-3}m^2$

$s_{wst} := 440mm$ $n_{wst} := 3$

Whole bridge

$t_{bridge} := 18in$ $y_b := 2m$ $h_b := \frac{h_s}{2} + h_{st,p} + t_p = 0.89m$

$I_{yb} := 0.01454b_b^4$ From grasshopper

$y_{tp} := 0.51819in$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 44\frac{kN}{m^2}$

$Q := q\cdot b_p = 154\,\frac{kN}{m}$

$M_{Ed} := \frac{Q\cdot t_{bridge}^2}{8} = 6.237\times10^3\,kN\,m$

$V_{Ed} := \frac{Q\cdot t_{bridge}}{2} = 1.386\times10^3\,kN$

Top plate

$\sigma_{x,Ed,tp} := \frac{M_{Ed}}{I_{yb}}\left(h_b - y_b - \frac{t_p}{2} - y_{tp}\right) = 157.255\,MPa$ $\sigma_{z,Ed,tp} = 0$

$S_{tp} := \frac{t_p b_p}{n_{st}+2}\left(h_b - y_b - \frac{t_p}{2} - y_{tp}\right) = 1.167\times10^{-3}m^3$

$\tau_{Ed,tp} := \frac{V_{Ed} S_{tp}}{I_{yb}\cdot\min(t_p, t_{wst})} = 27.798\,MPa$

Side plates

$\sigma_{x,Ed,sp} := \frac{M_{Ed}}{I_{yb}}\left(h_b - y_b - Y_{sp}\right) = 155.111\,MPa$

$\sigma_{z,Ed,sp} := \frac{Q}{n_s t_s} = 19.25\,MPa$

$S_{sp} := (h_b - y_b - Y_{sp})\cdot t_s\cdot\frac{(h_b - y_b - Y_{sp})}{2} + \frac{t_p b_p}{n_{st}+2}\left(h_b - y_b - \frac{t_p}{2} - Y_{sp}\right) = 9.622\times10^{-3}m^3$

$\tau_{Ed,sp} := \frac{V_{Ed} S_{sp}}{I_{yb}\cdot 4\cdot t_s} = 8.832\,MPa$

Governing Vondermeyde from FE-software

$\sigma_{x,1,UNIT} := 8.25\,\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

$\sigma_{x} := \sigma_{x,1,UNIT}\cdot q = 362.12\,MPa$ $^{0.0979}$ ^{MPa}

$\sigma_{xk} := \frac{(f_y)_0}{\sigma_V}\cdot 0.98\cdot\frac{I_{yb}\frac{b_w}{2}}{S_{tp}+S_{sp}}\left(h_b - y_b - \frac{h_w}{4} - Y_{tp}\right) + \frac{t_p b_p}{S_{tp}+S_{sp}}\left(h_b - y_b - \frac{t_p}{2} - Y_{tp}\right) = 1.483\times10^{-3}m^3$

$\sigma_{z,1,UNIT} := 35.20\,\frac{kN}{m^2}$ From a bucking eigenvalue analysis in Sofistik

$\sigma_{cr} := \frac{\sigma_{z,1,UNIT}}{q} = 0.802$

$\lambda_p := \sqrt{\frac{\sigma_{xk}}{\sigma_{cr}}} = 1.106$ Slenderness for all parts

Top plate check

$\eta_{fp} := 1$

$\rho_{p,tp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\eta_{fp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.724$

Plate-like behaviour [A.1]

$\sigma_{E,tp} := \frac{\pi^2 E t_p^2}{12\cdot(1-\nu^2)\cdot b_p^2} = 1.549\,MPa$

$I_{y,tp} := \frac{b_p t_p^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-7}m^4$

$y_{tp,el,tp} := \frac{A_p\frac{t_p}{2} + n_{st}A_{st}\left(t_p + \frac{h_{st}}{2}\right)}{A_p + n_{st}A_{st}} = 32.411\,mm$

$I_{y,tp} := \frac{b_p t_p^3}{12} + A_p\left(y_{tp,el,tp} - \frac{t_p}{2}\right)^2 + n_{st}\left[\frac{t_{wst} h_{st}^3}{12} + A_{st}\left(t_p + \frac{h_{st}}{2} - y_{tp,el,tp}\right)^2\right] = 1.052\times10^{-6}m^4$

$\gamma_{tp} := \frac{I_{y,tp}}{I_{y,tp}} = 328.143$

$k_{\sigma} := \frac{n_{st}A_{st}}{A_p} = 0.576$

$\alpha_{tp} := \min\left(\frac{\gamma_{tp}}{k_{\sigma}}, 0.5\right) = 0.571$

$k_{\sigma,p,tp} := \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\gamma_{tp}-1)\cdot(1+k_{\sigma})} & \text{if } \alpha_{tp} < \sqrt{\gamma_{tp}} \\ \frac{4\cdot(1+\sqrt{\gamma_{tp}})}{(\gamma_{tp}-1)\cdot(1+k_{\sigma})} & \text{if } \alpha_{tp} \ge \sqrt{\gamma_{tp}} \end{cases} = 639.127$

$\sigma_{cr,p,tp} := k_{\sigma,p,tp}\cdot\sigma_{E,tp} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1,eff,tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot b_{tp} = 0.175m$

$b_{2,eff,tp} := \frac{2}{5-\psi_{tp}}\cdot b_{tp} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{el,1,tp} := b_{1,eff,tp} + b_{2,eff,tp} = 0.35m$

$y_{tp,el,1,tp} := \frac{t_p b_{el,1,tp}\frac{t_p}{2} + A_{st}\left(t_p + \frac{h_{st}}{2}\right)}{t_p b_{el,1,tp} + A_{st}} = 0.034m$ combined center of gravity for stiffener and contributing top plate.

$I_{1,1,tp} := \frac{b_{el,1,tp} t_p^3}{12} + b_{el,1,tp} t_p\left(y_{tp,el,1,tp} - \frac{t_p}{2}\right)^2 + \frac{t_{wst} h_{st}^3}{12} + A_{st}\left(t_p + \frac{h_{st}}{2} - y_{tp,el,1,tp}\right)^2$ Combined moment of inertia

$\sigma_{cr,el,tp} := \frac{\pi^2 E I_{1,1,tp}}{A_{el,1,tp} b_{tp}^2} = 1.026\times10^3\,MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,p} = \sigma_{cr,el}$

$\sigma_{cr,c,tp} := \sigma_{cr,el,tp}$

Reduction factor for column-like behaviour

$\lambda_p := \sqrt{\frac{f_{y,1,tp}}{\sigma_{cr,el,tp}}} = 0.045m$

$e_{tp} := \max\left(\left(y_{tp,el,1,tp} - t_p - t_{st} + \frac{h_{st}}{2} - y_{tp,el,tp}\right), 0\right) = 0.046m$

$\alpha_{e,tp} := 0.49 + \frac{0.09}{\frac{h_{st}}{t_{tp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e,tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 1.375$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1\right) = 0.456$

Interpolation

$\zeta_{tp} := \min\left(\max\left(\frac{\sigma_{cr,tp,sp}}{\sigma_{cr,c,tp}} - 1.0\right), 1\right) = 0$

$\rho_{c,tp} := \begin{cases} \left[\left(\rho_{p,tp} - \chi_{tp}\right)\zeta_{tp}\left(2 - \zeta_{tp}\right) + \chi_{tp}\right]\cdot\sigma_{tp} & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right)\zeta_{tp}\left(2 - \zeta_{tp}\right) + \chi_{tp}\right]\sigma_{tp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right)\zeta_{tp}\left(2 - \zeta_{tp}\right) + \chi_{tp}\right]\sigma_{tp}}{\gamma_{M1}} \ge 1 \end{cases} = 0.456$

Shear

$\eta_t := 1.2$ 7.1 (2)

$\lambda_{tp} := \begin{cases} 0.76 & \text{if } \lambda_p < \frac{0.83}{\eta_t} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_t} \le \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 0.759$

$\frac{\rho_{c,tp}\sigma_{tp}}{\gamma_{M0}} = 0.447$

Side plates check

x-direction

$\sigma_{max} := \sigma_{x,Ed,sp}$

$\sigma_{E0} := \left[\frac{-\sigma_{max}}{(f_y)_0} + \sigma_{max}\right] - 1$

$\psi_{sp,x} := \frac{\sigma_{E}(h_b - y_b - h_s)}{\sigma_{E}(h_b - y_b)} = -0.076$

$\sigma_{x,sup} := \frac{M_{Ed}}{I_{yb}}(h_b - y_b - Y_{sp}) = 155.111\,MPa$

$\sigma_{x,Inf} := \frac{M_{Ed}}{I_{yb}}(h_b - y_b - h_s - Y_{sp}) = -11.746\,MPa$

$\psi_{sp1,x} := \max(3 + \psi_{sp,x}, 0) = 2.924$ $\psi_{sp1} := \frac{\sigma_{x,Inf}}{\sigma_{x,sup}} = -0.076$

$\rho_{p,sp,x} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(\psi_{sp1})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.773$

Plate-like behaviour

$\sigma_{E,sp,x} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot b_s^2} = 20.048\,MPa$

$\sigma_{cr,sp,x} := \sigma_{E,sp,x}\cdot k_{\sigma,sp,x} = 167.248\,MPa$

$k_{\sigma,sp,x} := \begin{cases} 1 & \text{if } \psi_{sp,x} \ge 1 \\ \frac{8.2}{(1.05 + \psi_{sp,x}^2)} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\cdot\psi_{sp,x} + 9.78\cdot\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98\cdot(1 - \psi_{sp,x})^2 & \text{if } -1 > \psi_{sp,x} > -3 \end{cases} = 8.342$

Column-like behaviour

$\sigma_{cr,c,sp,x} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot b_{sp}^2} = 0.759\,MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 1.206$

$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1\right) = 0.592$

Interpolation

$\zeta_{sp,x} := \min\left(\max\left(\frac{\sigma_{cr,sp,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0\right), 1\right) = 1$

$\rho_{c,sp,x} := \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right)\zeta_{sp,x}\left(2 - \zeta_{sp,x}\right) - \right]\cdot\sigma_{sp} & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\zeta_{sp,x}\left(2 - \zeta_{sp,x}\right) - \right]\sigma_{sp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\zeta_{sp,x}\left(2 - \zeta_{sp,x}\right) - \chi_{sp}\right]\sigma_{sp}}{\gamma_{M1}} \ge 1 \end{cases} = 0.773$

Z-direction

$\eta_{sp,z} := 1$

$\alpha_{p,sp} := 0.34$ $\chi_{p0,sp} := 0.7$

$\phi_{sp,sp} := 0.5\left[1 + \alpha_{sp,sp}\left(\chi_p - \chi_{p0,sp}\right) + \chi_p^2\right] = 1.122$

$\rho_{p,sp,z} := \begin{cases} 1 & \text{if } \chi_p < 0.673 \\ \frac{1}{\phi_{sp,sp} + \sqrt{\phi_{sp,sp}^2 - \chi_p^2}} & \text{if } \chi_p > 0.673 \end{cases} = 0.661$

Plate-like behaviour

$k_{\sigma,p,sp,z} := 4$

$\sigma_{E,sp,z} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot b_{sp}^2} = 0.759\,MPa$

$\sigma_{cr,p,sp,z} := k_{\sigma,p,sp,z}\cdot\sigma_{E,sp,z} = 3.037\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,z} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot b_{sp}^2} = 20.048\,MPa$

Interpolation

$\zeta_{sp,z} := \min\left(\max\left(\frac{\sigma_{cr,p,sp,z}}{\sigma_{cr,c,sp,z}} - 1.0\right), 1\right) = 0$

$\rho_{c,sp,z} := \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right)\zeta_{sp,z}\left(2 - \zeta_{sp,z}\right) - \right]\cdot\sigma_{sp} & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\zeta_{sp,z}\left(2 - \zeta_{sp,z}\right) - \right]\sigma_{sp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\zeta_{sp,z}\left(2 - \zeta_{sp,z}\right) - \chi_{sp}\right]\sigma_{sp}}{\gamma_{M1}} \ge 1 \end{cases} = 0.592$

Top plate stiffeners - check

x-direction

$\rho_{p,st,tp} := \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 0.751$

$\psi_{sp,x} := \frac{\sigma_{E}(h_b - y_b - h_{st})}{\sigma_{E}(h_b - y_b)} = 0.613$

$$\sigma_{E,cpt,x} := \frac{\pi^2 \cdot E \cdot t_{w}^{\,2}}{12 \cdot \left(1 - \nu^2\right) \cdot h_{w}^{\,2}} = 2.479 \cdot 10^{3}\ \text{MPa}$$

$$k_{\sigma,cpt,x} := \begin{cases} 1 & \text{if } \psi_{cpt,x} \geq 1 \\ \dfrac{8.2}{1.05 + \psi_{cpt,x}} & \text{if } 1 > \psi_{cpt,x} > 0 \\ 7.81 & \text{if } \psi_{cpt,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{cpt,x} + 9.78 \cdot \psi_{cpt,x}^{\,2} & \text{if } 0 > \psi_{cpt,x} > -1 \\ 23.9 & \text{if } \psi_{cpt,x} = -1 \\ 5.98 \cdot \left(1 - \psi_{cpt,x}\right)^2 & \text{if } -1 > \psi_{cpt,x} > -3 \\ 95.68 & \text{if } \psi_{cpt,x} \leq -3 \end{cases} = 4.01$$

$$\sigma_{cr,cpt,x} := k_{\sigma,cpt,x} \cdot \sigma_{E,cpt,x} = 9.941 \cdot 10^{3}\ \text{MPa}$$

Column-like behaviour

$$\sigma_{ccr,cpt,x} := \frac{\pi^2 \cdot E \cdot t_{w}^{\,2}}{12 \cdot \left(1 - \nu^2\right) \cdot h_{ip}^{\,2}} = 12.147\ \text{MPa}$$

$$\chi_{cpt} := \chi_{ip}$$

Interpolation

$$\xi_{cpt,x} := \min\left(\max\left(\frac{\sigma_{cr,cpt,x}}{\sigma_{ccr,cpt,x}} - 1.0\right), 1\right) = 1$$

$$\rho_{cpt,x} := \begin{cases} \left[\left(\rho_{ht,cpt} - \chi_{cpt}\right) \cdot \chi_{cpt} \cdot \left(2 - \xi_{cpt,x}\right) + \chi_{cpt}\right] & \text{if } \dfrac{\left[\left(\rho_{ht,cpt} - \chi_{cpt}\right) \cdot \chi_{cpt} \cdot \left(2 - \xi_{cpt,x}\right) + \chi_{cpt}\right] \cdot \sigma_{cpt}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{ht,cpt} - \chi_{cpt}\right) \cdot \chi_{cpt} \cdot \left(2 - \xi_{cpt,x}\right) + \chi_{cpt}\right] \cdot \sigma_{cpt}}{\gamma_{M1}} \geq 1 \end{cases} = 0.751$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,cpt}}{\frac{\rho_{x,cpt} \cdot f_y \cdot t_f}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,cpt}}{\frac{\chi_w \cdot f_y \cdot t_f}{\gamma_{M1}}}\right)^2 = 0.975$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,cpt}}{\frac{\rho_{x,cpt} \cdot f_y \cdot t_f}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,cpt}}{\frac{\rho_{z,cpt} \cdot f_y \cdot t_f}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,cpt}}{\frac{\rho_{x,cpt} \cdot f_y \cdot t_f}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z,Ed,cpt}}{\frac{\rho_{z,cpt} \cdot f_y \cdot t_f}{\gamma_{M1}}}\right) + 3 \cdot \left(\frac{\tau_{Ed,cpt}}{\frac{\chi_w \cdot f_y \cdot t_f}{\gamma_{M1}}}\right)^2 = 0.819$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,cpt}}{\frac{\rho_{x,cpt} \cdot f_y \cdot t_{sl}}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,cpt}}{\frac{\chi_w \cdot f_y \cdot t_{sl}}{\gamma_{M1}}}\right)^2 = 0.342$$

EWM - Changed side plate thickness

This chapter covers the EWM when changing the side plate thickness to 6 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s.sd} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{su} := \frac{b_d}{n_{s.su}} = 350\,mm$

Side plate: $h_s := 389.2mm$ $t_s := 6mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140mm$ $t_{su} := 16mm$ $a_{su} := 6mm$

Stiffeners under: $h_{sl} := 120mm$ $t_{sl} := 12mm$ $a_{sl} := 5mm$

$h_{sl.y} := h_{sl} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{tt} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

$A_d := t_d \cdot b_d$ $A_s := 2 \cdot t_s \cdot h_s$ $A_w := 2 \cdot t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$ $A_{sl} := n_{sl} \cdot t_{sl} \cdot h_{sl}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.117\,m^2$

$A_c := A_{tot} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$f_y := 355MPa$

$z_{tp} :=$ (distance to centre of gravity)

$$\frac{A_d \cdot \frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_b\left(t_d + h_s + h_{wy} + \frac{t_b}{2}\right) + A_{sl}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{sl.y}}{2}\right]}{A_{tot}} = 369.443\,mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206\,mm$

Stiffeners overlunder:

$$I_{y.st.su} := n_{su} \cdot \left[\frac{t_{su} \cdot h_{su}^3}{12} + t_{su} \cdot h_{su} \cdot \left(t_d + \frac{h_{su}}{2} - z_{tp}\right)^2\right] = 1.722 \times 10^{-7}\,m^4$$

$I_{y.st.su.1} := 6 \cdot 1.506\,10^{-8}\,m^4$ from FEM

$$I_{y.st.sc} := I_{y.st.su.1} + 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{sl.y}}{2} - z_{tp}\right]^2 \ldots = 8.847 \times 10^{-4}\,m^4$$
$$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{sl.y}}{2} - z_{tp}\right]^2 \ldots$$
$$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{sl.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312\,m^4$ from FEM

$$I_{y.w} := I_{y.w.1} + 2 \cdot t_w \cdot h_w \cdot \left[h_b - \frac{h_b}{2} + \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 2.989 \times 10^{-3}\,m^4$$

Deck:

$$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.649 \times 10^{-7}\,m^4$$

Bottom flange:

$$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_b - z_{tp} - \frac{t_b}{2}\right)^2 = 4.204 \times 10^{-7}\,m^4$$

Side plate:

$$I_{y.s} := 2 \cdot \frac{t_s \cdot h_s^3}{12} + 2 t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 1.859 \times 10^{-4}\,m^4$$

Total:

$I_{y.tot} := I_{y.st.su} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses

Stresses

$\sigma_{max} := f_y / \gamma_{M0} = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$$k_{\sigma.4.2s}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2l}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left(\frac{\lambda_p - 0.055 \cdot \max(3 + \psi, 0)}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.083$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.398$

$s_{s.so} := s_{su} - t_{su}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.969$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 0.861$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.03\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 359.443\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s\left(h_{s.c}\right) = 309.591\,mm$

$h_{s.eff.c1} := 0.4\,h_{s.eff.c} = 123.837\,mm$

$h_{s.eff.c2} := 0.6\,h_{s.eff.c} = 185.755\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 339.348\,mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 3.715 \times 10^{-3}\,mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right) \cdot t_s = 4.072 \times 10^{-3}\,mm^2$

Critical buckling stress, orthotropic plate:

$$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12\left(1 - \nu^2\right) \cdot h_s^2} = 378.806\,MPa$$ EC1993-1-5 eq A.1

Stiffeners

$z_{su} := t_d = 10\,mm$

$z_{sl} := t_d + h_{su} = 150\,mm$

$\psi_{su} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.611$

$k_{\sigma,4.2|4}(\psi_{so}) = 0.608$ Higher stress close to support

Class4 := "Class4" if $\frac{b_{so}}{t_{so}} \geq 21\cdot\varepsilon\cdot(\gamma_{so})\cdot\sqrt{k_{\sigma,4.2|4}(\psi_{so})}$ = "Lower than class 4"
"Lower than class 4" otherwise

$\lambda_{p,so} := \lambda_p(k_{\sigma,4.2|4}(\psi_{so}), b_{so}, t_{so}) = 0.486$

$\rho_{d,so} := \rho_{4.2}(\lambda_{p,so}, \psi_{so}) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so,eff} := \rho_{d,so}\cdot b_{so} = 140\cdot mm$

$A_{so,eff} := n_{so}\cdot t_{so}\cdot b_{so,eff} = 2.016\times 10^{4}\cdot mm^{2}$

Deck

$\frac{b_{so}}{t_d} = 35$ $42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ := "Class4" if $\frac{b_{so}}{t_d} \geq 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4"
"Lower than class 4" otherwise

$z_d := -\frac{t_d}{2} = -5\times 10^{-3}\cdot m$ $\psi_d := \psi_{neg}(t_d, z_d) = 1$

Assume: $n_{so} := n_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma,d} := k_{\sigma,4.1}(\psi_d) = 4$

$\lambda_{p,d} := \lambda_p(k_{\sigma,d}, b_{so}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p,d}, \psi_d) = 0.962$ Internal cross-sectional parts

$y_{so,eff} := \rho_d\cdot y_{c,so} = 321.461\cdot mm$

Areas:

$A_{so} := n_{so}\cdot b_{so}\cdot t_d + n_{so}\cdot t_{so}\cdot(b_{so,eff} + t_d) = 5.31\times 10^{4}\cdot mm^{2}$

$A_{c,eff,loc} := n_{so}\cdot b_{so,eff}\cdot t_d + n_{so}\cdot t_{so}\cdot(b_{so,eff} + t_d) = 5.053\times 10^{4}\cdot mm^{2}$

More than three longitudinal stiffeners --> $k_{\sigma,p}$ obtained with eq. A.2

$I_p := \frac{b_d\cdot t_d^{3}}{12\cdot(1-\nu^{2})} = 3.205\times 10^{-5}\cdot mm^{4}$

$y_{tp,sl} := \frac{n_{so}\cdot t_{so}\cdot b_{so}\cdot\left(t_d + \frac{b_{so}}{2}\right) + b_d\cdot t_d\cdot\frac{t_d}{2}}{n_{so}\cdot t_{so}\cdot b_{so} + b_d\cdot t_d} = 32.411\cdot mm$

$I_{sl} := \left[\frac{b_d\cdot t_d^{3}}{12} + b_d\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp,sl}\right)^{2}\right] + \left[\frac{n_{so}\cdot t_{so}\cdot b_{so}^{3}}{12} + n_{so}\cdot b_{so}\cdot t_{so}\cdot\left(t_d + \frac{b_{so}}{2} - y_{tp,sl}\right)^{2}\right] = 1.052\times 10^{8}\cdot mm^{4}$

$\gamma := \frac{I_{sl}}{I_p} = 328.143$ $\alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so}\cdot b_{so}\cdot t_{so} = 0.02\cdot m^{2}$ EC1993-1-5A.1 (2)

$A_p := b_d\cdot t_d = 0.035\cdot m^{2}$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma,p} := \frac{2\cdot\left[(1+\alpha^{2})^{2} + \gamma - 1\right]}{\alpha^{2}\cdot(\psi+1)\cdot(1+\delta)}$ if $\alpha \leq \sqrt{\gamma}$ $= 639.127$
$\frac{4\cdot\left(1+\sqrt{\gamma}\right)}{(\psi+1)\cdot(1+\delta)}$ otherwise

Critical buckling stress, orthotropic plate:

Effective area of the stiffened plate(deck):

$A_{c,eff} := \rho_{4.2}(\lambda_p, \psi)\cdot A_{c,eff.loc} + n_{so}\cdot t_d = 5.375\times 10^{4}\cdot mm^{2}$ 3-1-5 eq.A.1

Buckling coefficient: $\chi_{A,c} := \frac{A_c}{A_g} = 0.952$

$N_{Rd} := A_{c,eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 1.908\times 10^{4}\cdot kN$

$\sigma_{cr,p} :=$

Column like behaviour

$y_{tp,sl.c} := \frac{t_{so}\cdot b_{so}\cdot\left(t_d + \frac{b_{so}}{2}\right) + n_{so}\cdot t_d\cdot\frac{t_d}{2}}{t_{so}\cdot b_{so} + n_{so}\cdot t_d} = 34.268\cdot mm$

$I_{sl.c} := \left[\frac{n_{so}\cdot t_d^{3}}{12} + n_{so}\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp,sl.c}\right)^{2}\right] + \left[\frac{t_{so}\cdot b_{so}^{3}}{12} + b_{so}\cdot t_{so}\cdot\left(t_d + \frac{b_{so}}{2} - y_{tp,sl.c}\right)^{2}\right] = 1.137\times 10^{-4}\cdot mm^{4}$

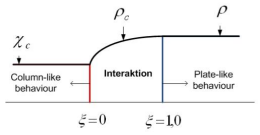
Buckling resistance stiffener

$A_{sl} := t_{so}\cdot b_{so} + n_{so}\cdot t_d = 5.74\times 10^{3}\cdot mm^{2}$

$A_{sl,eff} := t_{so}\cdot b_{so,eff} + n_{so}\cdot t_d = 5.455\times 10^{3}\cdot mm^{2}$

$\beta_c := \frac{A_{sl,eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr,sl} := \frac{\pi^{2}\cdot E\cdot I_{sl.c}}{A_{sl}\cdot a^{2}} = 1.026\times 10^{3}\cdot MPa$

Slenderness ratio

$\lambda_{sc} := \sqrt{\frac{\beta_c\cdot f_y(t_{sl})}{\sigma_{cr,sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\cdot mm$

$\bar{a} := \min\left(\left[\left(y_{tp,sl} - \frac{t_d}{2}\right), \left(t_d + \frac{b_{so}}{2} - y_{tp,sl}\right)\right]\right) = 47.589\cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{\bar{a}}} = 0.522$

$\phi := 0.5\cdot\left[1 + \alpha_e\cdot(\lambda_c - 0.2) + \lambda_c^{2}\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^{2} - \lambda_c^{2}}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr,p}}{\sigma_{cr,sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c)\cdot\xi\cdot(2-\xi) + \chi_c = 0.792$

Checks

Mean stress

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{axsl} := n_{so}\cdot s_{so.eff}\cdot t_d\cdot\sigma_{neg}\left(t_d + \frac{t_d}{2}\right) + A_{so}\cdot\sigma_{neg}\left(t_d + \frac{b_{so.eff}}{2}\right) = -1.574\times 10^{4}\cdot kN$

$\sigma_{com.Ed} := \frac{F_{axsl}}{n_{so}\cdot s_{so.eff}\cdot t_d + A_{so}} = -320.6\cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c\cdot f_y(t_{so})}{\gamma_M} = 281.032\cdot MPa$

$K_{A.2.1} :=$ "OK" if $\frac{\rho_c\cdot f_y(t_{so})}{\gamma_M} > \sigma_{com.Ed}$ = "OK"
"EJ OK" otherwise

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{so}\cdot t_d\cdot s_{so.eff} = 0.032\cdot m^{2}$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.113\cdot m^{2}$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{s.eff}\cdot\frac{t_d}{2} + A_{so.eff}\cdot\left(t_d + \frac{b_{so.eff}}{2}\right) - \\ &+ 2\cdot\left[t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2}\right) - \right. \\ &\left. + t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2}\right) + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2}\right)\right] - \\ &+ A_w\cdot\left(t_d + b_s + \frac{b_{wy}}{2}\right) + A_{su}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(b_{wy} - s_{sub}) - \frac{b_{su.t}}{2}\right] - \\ &+ A_b\cdot\left(b_w - \frac{b_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 379.747\cdot mm$

$I_{y.s.eff} := 2\cdot\left[\frac{t_s\cdot b_{s.eff.c1}^{3}}{12} + t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^{2} - \right.$
$+ \frac{t_s\cdot b_{s.eff.c2}^{3}}{12} + t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^{2} -$
$\left. + \frac{t_s\cdot b_{s.t}^{3}}{12} + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2} - z_{tp.eff.neg}\right)^{2}\right] = 1.729\times 10^{-4}\cdot m^{4}$

$I_{y.d.eff} := \frac{n_{so}\cdot s_{so.eff}\cdot t_d^{3}}{12} + n_{so}\cdot s_{so}\cdot t_d\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^{2} = 4.515\times 10^{-3}\cdot m^{4}$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.d.eff} = 0.015\cdot m^{4}$

Negative moment

Effective cross-section with negative moment(lower flange in tension)

Area

$A_{tot.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053\times 10^{4}\cdot mm^{2}$

$A_{c.eff.loc.mod} := \frac{\rho_c\cdot f_y(t_d)\cdot A_{sl}}{\sigma_{com.Ed}\cdot\gamma_M} = -5.032\times 10^{3}\cdot mm^{2}$ (A.3)

$A_{tot.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

effective in the cross-section: $A_{so.eff.mod} + n_{so.eff}\cdot t_d$ if $K_{A.2.1}$ = "EJ OK" $= 432.173\cdot cm^{2}$

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.041\times 10^{3}\cdot cm^{2}$

A_{su} Lower stiffener in tension -> no reduction

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with reduction factors.

$A_{c.eff.loc.mod} = 5.053\times 10^{4}\cdot mm^{2}$

$t_{d.eff} := \begin{cases} \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot t_d & \end{cases} = 7.916\cdot mm$

Effective areas, effective centre of gravity and

$y_{tp.eff.neg} := \frac{\begin{aligned} &n_{so}\cdot b_{so.eff}\cdot t_{so}\cdot\frac{t_{d.eff}}{t_d}\cdot\left(t_d + \frac{b_{so}}{2}\right) + s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} - \\ &+ n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_d}{2}\right) + t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2}\right) - \\ &+ t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2}\right) + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2}\right) - \\ &+ A_w\cdot\left(t_d + b_s + \frac{b_{wy}}{2}\right) + A_{su}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(b_{wy} - s_{sub}) - \frac{b_{su.t}}{2}\right] - \\ &+ A_b\cdot\left(b_w - \frac{b_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 405.253\cdot mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so}\cdot s_{so.eff}\cdot t_d^{3}}{12} + t_d\cdot n_{so}\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^{2} = 5.15\times 10^{-3}\cdot m^{4}$

Assume: $s_{sub} := \min\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{sub} = 7.206\cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so}\cdot\frac{t_{so}\cdot b_{so.eff}^{3}}{12} + n_{so}\cdot t_{so}\cdot b_{so.eff}\cdot\left(t_d + \frac{b_{so.eff}}{2} - z_{tp.eff.neg}\right)^{2} = 2.166\times 10^{-3}\cdot m^{4}$

$I_{y.st.su.1.fin} = 6\cdot 1.506\cdot 10^{-6}\cdot m^{4}$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} - = 7.158\times 10^{-4}\cdot m^{4}$
$+ 2\cdot t_{su}\cdot b_{su}\cdot\left[t_d + b_s + \frac{1}{3}\cdot(b_{wy} - s_{suby}) - \frac{b_{su.y}}{2} - z_{tp.eff.neg}\right]^{2} -$
$+ 2\cdot t_{su}\cdot b_{su}\cdot\left[t_d + b_s + \frac{2}{3}\cdot(b_{wy} - s_{suby}) - \frac{b_{su.y}}{2} - z_{tp.eff.neg}\right]^{2} -$
$+ 2\cdot t_{su}\cdot b_{su}\cdot\left[t_d + b_s + (b_{wy} - s_{suby}) - \frac{b_{su.y}}{2} - z_{tp.eff.neg}\right]^{2}$

Web:

$I_{y.w.1.fin} := 2\cdot 0.000312\cdot m^{4}$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2\cdot t_w\cdot b_w\cdot\left[b_w - \frac{b_b}{2} - \frac{1}{2}\cdot(b_{wy}) - z_{tp.eff.neg}\right]^{2} = 2.405\times 10^{-3}\cdot m^{4}$

Bottom flange:

$I_{y.b.fin} := \frac{b_b\cdot t_b^{3}}{12} + t_b\cdot b_b\cdot\left(b_w - z_{tp.eff.neg} - \frac{t_b}{2}\right)^{2} = 3.637\times 10^{-3}\cdot m^{4}$

Side plate:

$I_{y.s.fin} := 2\cdot\left[\frac{t_s\cdot b_{s.eff.c1}^{3}}{12} + t_s\cdot b_{s.eff.c1}\cdot\left(t_d + \frac{b_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^{2} - \right.$
$+ \frac{t_s\cdot b_{s.eff.c2}^{3}}{12} + t_s\cdot b_{s.eff.c2}\cdot\left(t_d + b_{s.c} - \frac{b_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^{2} -$
$\left. + \frac{t_s\cdot b_{s.t}^{3}}{12} + t_s\cdot b_{s.t}\cdot\left(t_d + b_s - \frac{b_{s.t}}{2} - z_{tp.eff.neg}\right)^{2}\right] = 2.105\times 10^{-4}\cdot m^{4}$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.428\times 10^{10}\cdot mm^{4}$

Load

Load capacity:

$q_{.M} := 88\frac{kN}{m^{2}}$ The load capacity is calculated with different checks. The load capacity is $q_{.M}$ when checking the moment capacity.

$Q_{.M} := q_{.M}\cdot b_d = 308\frac{kN}{m}$

$M_{Ed} := \frac{Q_{.M}\cdot l_{bridge}^{2}}{8} = 1.247\times 10^{4}\cdot kN\cdot m$

$q_{.V} := 58\frac{kN}{m^{2}}$ The load capacity is q_v when checking the shear capacity.

$Q_{.V} := q_{.V}\cdot b_d = 203\frac{kN}{m}$

$V_{Ed} := \frac{Q_{.V}\cdot l_{bridge}}{2} = 1.827\times 10^{3}\cdot kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.035\,m^3$

$M_{Rd} := W_{y.eff}\frac{f_d(f_y)}{\gamma_{M1}} = 1.251 \times 10^4\,kN\cdot m$

Moment capacity

$\frac{M_{Ed}}{M_{Rd}} = 0.997$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot t(t_w) \cdot t_w = 0.293\,m$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right)\frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{wall}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) + \frac{t_{ins}}{2} - \frac{\sin\left(\alpha_w\frac{\pi}{180}\right)\cdot h_{ins}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{ws}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{ws}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) = 1.324\,m$

Moment of inirta stiffeners z-direction

$I_{st.z} := 2\cdot t_{wt}\cdot h_{wt}\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$a_{mwb} := \frac{b_b}{2} + s_{wall}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) + \frac{t_{ins}}{2} = 0.525\,m$

$a_{mwb} := a_{st.z.1} + s_{ws}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) = 0.935\,m$

$a_{mwb} := a_{st.z.2} + s_{ws}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) = 1.346\,m$

Moment of inirta of 15st of the web z-direction

$I_{stw.z} := 2h_{web}\cdot t_w\cdot\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00\left(\frac{b_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{b_{s.eff}} \geq 1 \\ 4.00 + 5.34\left(\frac{b_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{b_{s.eff}} < 1 \end{cases} = 5.455$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when α_{wt}=0

$\lambda_{w1} := \frac{b_{s.eff}}{37.4\cdot t_s\cdot\varepsilon(f_y)\sqrt{k_{\tau 1}}} = 0.796$ EC1-3-5 eq 5.6

Shear buckling coefficient, web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.493$

$k_{\tau 2} := \max\left[\left(\frac{h_w}{a}\right)^2\sqrt{9\cdot\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)^3\cdot h_w\right)}}, \frac{2.1}{\min\left(t_s, t_w\right)}\sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 2.275 \times 10^4$ 3N 1993-1-5 (A.5)

SS-EN 1993-1-5 A.3 (1) och (2) ger:

Check := $\begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \leq 31\frac{\varepsilon(f_y)}{\eta}\sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases}$ = "Reduce"

$= 1.268 \times 10^4$

Reduce --> Shear buckling check is needed. EC1993-1-5 Ch. 5.1(2)

$\lambda_w := \max\left(\frac{h_w}{37.4\cdot t_w\cdot\varepsilon(f_y)\sqrt{k_{\tau 2}}}, \lambda_{w1}\right) = 0.796$ $2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3\cdot h_{web}}}$ otherwise SS-EN 1993-1-5 5.3 (5.6)

Check if shear buckling check is needed. EC1993-1-5 Ch. 5.1(2) --> shear buckling check is needed. Check has to be performed for stiffened webs:

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.043$ SS-EN 1993-1-5 5.3 Table 5.1, stiff end stiffener.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ s.583 bärande konstruktioner

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w\cdot f_d(f_y)\cdot h_{web}\cdot t_s}{\sqrt{3}\cdot\gamma_{M1}\cdot\gamma_\eta} = 923.486\,kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 1.827 \times 10^3\,kN$

$\frac{V_{Ed}}{2V_{bw.Rd}} = 0.989$ (contribution from two web)

$\frac{V_{Ed}}{V_{b.Rd}} < \frac{\eta\cdot f_d(f_y)\cdot h_{web}\cdot t_s}{\sqrt{3}\cdot\gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4, S83 bärande konstruktioner

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{tf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 400.253\,mm$

$z_{tf.eff.neg} := h_b - \frac{h_b}{2} - z_{tp.eff.neg} = 476.747\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg}\left|z_{tf.eff.neg}\right|\cdot f_d(f_y)}{\gamma_{M0}} \ldots = 7.276 \times 10^3\,kN\cdot m$
$+ \frac{A_{bf.eff.neg}\left|z_{bf.eff.neg}\right|\cdot f_d(f_y)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tf} := z\cdot\left[0.25 + \frac{1.6\cdot b_f\cdot t_d^2\cdot f_d(f_y)}{t_s\left(h_s + h_{wy}\right)^2\cdot f_y(f_y)}\right] = 0.745\,m$

$c_{tf} := z\cdot\left[0.25 + \frac{1.6\cdot b_b\cdot t_b^2\cdot f_d(f_y)}{t_s\left(h_s + h_{wy}\right)^2\cdot f_y(f_y)}\right] = 0.68\,m$

$V_{bf.Rd.cred} := \frac{b_f\cdot t_d^2\cdot f_d(f_y)}{c_{tf}\cdot\gamma_{M1}\cdot\gamma_\eta} + \frac{b_b\cdot t_b^2\cdot f_d(f_y)}{c_{tf}\cdot\gamma_{M1}\cdot\gamma_\eta} = 250.336\,kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cred}\left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} = 0\,kN \\ 0 & \text{otherwise} \end{cases}$

Shear capacity stiffened web:

$\frac{V_{bw.Rd} + V_{bf.Rd}}{V_{b.Rd}} < \frac{\eta\cdot f_d(f_y)\cdot h_{web}\cdot t_s}{\sqrt{3}\cdot\gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 923.486\,kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2V_{b.Rd}} = 0.989$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.787 \times 10^{-3}\,m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{ws.eff}\cdot t_d + n_{ws}\cdot s_{ws.eff}\cdot t_{d.eff} + n_{ws}\cdot t_{ws}\cdot t_{d.eff} + A_b = 432.579\,cm^2$

z_n for the flanges effective areas:

$z_{tp.f.eff.neg} := \frac{s_{ws.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{ws}\cdot s_{ws.eff}\cdot t_{d.eff}\left(\frac{t_{d.eff}}{2}\right) + n_{ws}\cdot t_{ws}\cdot t_{d.eff}\left(\frac{t_{d.eff}}{2}\right) + A_b\left(h_w - \frac{h_b}{2}\right)}{A_{eff.f.tot}} = 328.801\,mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.027\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff}\frac{f_d(f_y)}{\gamma_M} = 9.487 \times 10^3\,kN\cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.251 \times 10^4\,kN\cdot m$

Moment capacity check when $q_{-}M = 88\,\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.997$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_{-}V = 50\,\frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.989$

The Reduced stress method

-Changed side plate thickness to 6 mm

Material

$f_{y0} := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ 335\,MPa & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$E = 210\,GPa$

$\nu = 0.3$ $G = \frac{E}{2\cdot(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$t_b = 16\,mm$ $b_b = 1000\,mm$ $A_b = b_b t_b = 0.016\,m^2$

Top plate

$t_{tp} = 10\,mm$ $b_{tp} = 3500\,mm$ $A_{tp} = b_{tp} t_{tp} = 0.035\,m^2$

Side plates

$t_s = 6\,mm$ $h_s = 389.2\,mm$ $A_s = h_s t_s = 2.335 \cdot 10^{-3}\,m^2$

Webs

$t_w = 12\,mm$ $h_w = 1340\,mm$ $A_w = h_w t_w = 0.016\,m^2$

$h_{w,n} = 1250\,mm$ $h_{w,p} = 482.8\,mm$

Top stiffeners

$t_{st} = 16\,mm$ $h_{st} = 140\,mm$ $A_{st} = h_{st} t_{st} = 2.24 \cdot 10^{-3}\,m^2$

$s_{st} = 350\,mm$ $n_{st} = 9$

Web stiffeners

$t_{sw} = 12\,mm$ $h_{sw} = 120\,mm$ $A_{sw} = h_{sw} t_{sw} = 1.44 \cdot 10^{-3}\,m^2$

$s_{sw} = 440\,mm$ $n_{sw} = 3$

Whole bridge

$L_{bridge} = 18\,m$ $y_b = \frac{h_b}{2} + h_{w,p} + h_s + t_{tp} = 0.89\,m$ $y_b = 2\,m$

$I_{yb} = 0.014613\,m^4$ From grasshopper

$V_{yb} = 0.520476\,m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q = 64\,\frac{kN}{m}$

$Q = q\,L_{bridge} = 224\,\frac{MN}{m}$

$M_{Ed} = \frac{Q\,L_{bridge}}{8} = 9.072 \cdot 10^3\,kN\,m$

$V_{Ed} = \frac{Q\,L_{bridge}}{2} = 2.016 \cdot 10^3\,kN$

Top plate

$\sigma_{x,Ed,tp} = \frac{M_{Ed}}{I_{yy}}\left(h_w - y_b - \frac{t_{tp}}{2} - V_{tp}\right) = 226.303\,MPa$ $\sigma_{z,Ed,tp} = 0$

$S_{tp} = \frac{t_{tp} t_{tp}}{n_{st}+2}\left(h_w - y_b - \frac{t_{tp}}{2} - V_{tp}\right) = 1.16 \cdot 10^{-3}\,m^3$

$\tau_{Ed,tp} = \frac{V_{Ed} S_{tp}}{I_{yy} \min(t_s; t_{sb})} = 26.669\,MPa$

Side plates

$\sigma_{x,Ed,sp} = \frac{M_{Ed}}{I_{yy}}\left(h_w - y_b - V_{tp}\right) = 223.199\,MPa$

$\sigma_{z,Ed,sp} = \frac{Q}{A_b} = 18.667\,MPa$

$S_{sp} = \left(h_w - y_b - V_{tp}\right) t_s \frac{\left(h_w - y_b - V_{tp}\right)}{2} + \frac{t_{tp} t_{tp}}{n_{st}+2}\left(h_w - y_b - \frac{t_{tp}}{2} - V_{tp}\right) + \frac{h_{sw}}{n_{sw}+1} t_{st} t_s \left(h_w - y_b - \frac{h_{sw}}{2} - V_{tp}\right) = 9.685 \cdot 10^{-3}\,m^3$

$\tau_{Ed,sp} = \frac{V_{Ed} S_{sp}}{I_{yy} 2 t_s} = 12.779\,MPa$

Governing eigenvalue from FE-software

$\sigma_{x,LNA} = 5.482\,\frac{MN}{m^2}$ 411.349 MPa From a linear stress analysis in Sofistik

$\sigma_{tp} = \sigma_{x,LNA}\left(1 - \frac{y_b - V_{tp}}{y_b}\right) = 372.48\,MPa = 223.199\,MPa$ 5.9173

$\alpha_{cr,tb} = \frac{\sqrt{f_{y0}}}{\sigma_{x,tb}} = 0.953$ $\frac{t_b t_{tb}}{2}\left(h_w - y_b - \frac{h_{tb}}{4} - V_{tp}\right) + \frac{t_b t_{tb}}{n_{st}+2}\left(h_w - \frac{t_{tp}}{2} - V_{tp}\right) = 1.482 \cdot 10^{-3}\,m^3$

$n_{st} = 2$

$\sigma_{x,LNA,1} = 912.767\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} = \frac{\sigma_{x,LNA,1}}{\epsilon} = 1.606$

$\lambda_p = \sqrt{\frac{f_{y0}}{\sigma_{cr}}} = 0.77$ Slenderness for all parts

Top plate check

$\psi_{tp} = 1$

$\rho_{p,tp} = \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\,(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.927$

Plate-like behaviour [A.1]

$\sigma_{E,tp} = \frac{\pi^2 E t_{tp}^2}{12\left(1-\nu^2\right) b_{tp}^2} = 1.549\,MPa$

$I_{y,tp} = \frac{b_{tp} t_{tp}^3}{12\left(1-\nu^2\right)} = 3.205 \cdot 10^{-7}\,m^4$

$y_{tp,sl,tp} = \frac{A_{sl}\frac{t_{tp}}{2} + n_{st} A_{st}\left(t_{tp} + \frac{h_{st}}{2}\right)}{A_{sl} + n_{st} A_{st}} = 32.411\,mm$

$I_{sl,tp} = \frac{b_{tp} t_{tp}^3}{12} + A_{sl}\left(y_{tp,sl,tp} - \frac{t_{tp}}{2}\right)^2 + n_{st}\left[\frac{t_{st} h_{st}^3}{12} + A_{st}\left(t_{tp} + \frac{h_{st}}{2} - y_{tp,sl,tp}\right)^2\right] = 1.052 \cdot 10^{-6}\,m^4$

$n_{st} = 9$

$\gamma_{tp} = \frac{I_{sl,tp}}{I_{y,tp}} = 328.143$

$\delta_{tp} = \frac{A_{st} n_{st}}{A_{sl}} = 0.576$

$\alpha_{tp} = \min\left(\frac{\gamma_{tp}}{6\,\delta_{tp}}; 0.5\right) = 0.571$ $\sqrt{\gamma_{tp}} = 4.256$

$k_{\sigma,p,tp} = \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 \sqrt{\gamma_{tp}} - \gamma_{tp} - 1\right]}{\alpha_{tp}^2\left(\alpha_{tp}-1\right)\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} \sqrt{\gamma_{tp}} \\ \frac{4\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\alpha_{tp}-1\right)\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} \sqrt{\gamma_{tp}} \end{cases} = 639.127$

$\sigma_{cr,p,tp} = k_{\sigma,p,tp} \sigma_{E,tp} = 990.256\,MPa$

Column-like behaviour [A.5.3]

$b_{1,sl,tp} = \frac{3-\psi_{tp}}{5-\psi_{tp}} b_{tp} = 0.175\,m$

$b_{2,sup,tp} = \frac{2}{5-\psi_{tp}} b_{tp} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{sl,1,tp} = b_{1,sl,tp} + b_{2,sup,tp} = 0.35\,m$

$y_{tp,sl,1,tp} = \frac{t_{tp} b_{sl,1,tp}\frac{t_{tp}}{2} + A_{st}\left(t_{tp} + \frac{h_{st}}{2}\right)}{t_{tp} b_{sl,1,tp} + A_{st}} = 0.034\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl,1,tp} = \frac{b_{sl,1,tp} t_{tp}^3}{12} + b_{sl,1,tp} t_{tp}\left(y_{tp,sl,1,tp} - \frac{t_{tp}}{2}\right)^2 + \frac{t_{st} h_{st}^3}{12} + A_{st}\left(t_{tp} + \frac{h_{st}}{2} - y_{tp,sl,1,tp}\right)^2$ Combined moment of inertia

$\sigma_{cr,sl,tp} = \frac{\pi^2 E I_{sl,1,tp}}{A_{sl,1,tp}\, y_{tb}^2} = 1.026 \cdot 10^3\,MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,sl} = \sigma_{cr,pl}$

$\sigma_{cr,c,tp} = \sigma_{cr,sl,tp}$

Reduction factor for column-like behaviour

$\lambda_{tp} = \sqrt{\frac{A_{sl,1,tp} f_{y0}}{\sigma_{cr,sl,tp}}} = 0.045\,m$

$e_{tp} = \max\left(e_{tp,sl,1,tp} - y_b t_{tp} + \frac{h_{sw}}{2} - y_{tp,sl,1,tp}\right) = 0.046\,m$

$\alpha_{e,tp} = 0.49 - \frac{0.09}{\frac{2e}{h_{tp}}} = 0.582$ Stiffener with open cross section (curve c)

$\phi_{tp} = 0.5\left[1 + \alpha_{e,tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.963$ [6.3.1.2]

$\chi_{tp} = \min\left(\frac{1}{\left(\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}\right)}; 1\right) = 0.649$

Interpolation

$t_{tp} = \min\left(\max\left(\frac{\sigma_{cr,tp,tp}}{\sigma_{cr,sl,tp}} - 1.0\right); 1\right) = 0$

$\rho_{c,tp} = \begin{cases} \left[\left(\rho_{p,tp} - \chi_{tp}\right)\chi_{tp}\left(2 - t_{tp}\right) - t_{tp}\right] \frac{\gamma_{tp}}{\gamma_{M1}} & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right)\chi_{tp}\left(2 - t_{tp}\right) - t_{tp}\right]\gamma_{tp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right)\chi_{tp}\left(2 - t_{tp}\right) - t_{tp}\right]\gamma_{tp}}{\gamma_{M1}} > 1 \end{cases} = 0.649$

Shear

$\eta = 1.2$ 7.1 (2)

$\chi_w = \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\lambda_p} < \eta \\ \frac{1.37}{\left(0.7 + \lambda_p\right)} & \text{if } \lambda_p > 1.08 \end{cases} = 1.077$

$\frac{\tau_{cp,tp} \gamma_{tp}}{\gamma_{M1}} = 0.619$

Side plates check

x-direction

$\sigma_{max} = f_y(\psi)$

$\psi_{s1} = \left[\frac{-\sigma_{min}}{f(\psi)} + \sigma_{max}\right] - 1$

$\psi_{sp,x} = \frac{\sigma\left(h_w - t_{tp} - h_s\right)}{\sigma\left(h_w - t_{tp}\right)} = -0.083$

$\sigma_{x,top} = \frac{M_{Ed}}{I_{yy}}\left(h_w - t_{tp} - V_{tp}\right) = 223.199\,MPa$

$\sigma_{x,bot} = \frac{M_{Ed}}{I_{yy}}\left(h_w - t_{tp} - h_s - V_{tp}\right) = -18.423\,MPa$

$\psi_{sp1,x} = \max\left(3 + \psi_{sp,x}; 0\right) = 2.917$ $\psi_{sp1} = \frac{\sigma_{x,bot}}{\sigma_{x,top}} = -0.083$

$\rho_{p,s,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\left(\psi_{sp1,x}\right)}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1.028$

Plate-like behaviour

$\sigma_{E,sp,x} = \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) h_s^2} = 45.108\,MPa$

$\sigma_{cr,sp,x} = \sigma_{E,sp,x} \rho_{sp,x} = 378.719\,MPa$

$k_{\sigma,sp,x} = \begin{cases} 1 & \text{if } \psi_{sp,x} \ge 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp,x}\right)} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\,\psi_{sp,x} + 9.78\,\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98\left(1 - \psi_{sp,x}\right)^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 35.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 8.396$

Column-like behaviour

$\sigma_{cr,c,sp,x} = \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) h_w^2} = 1.708\,MPa$

$\alpha_{sp} = 0.21$

$\phi_{sp} = 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.857$

$\chi_{sp} = \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)}; 1\right) = 0.812$

Interpolation

$t_{sp,x} = \min\left(\max\left(\frac{\sigma_{cr,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0\right); 1\right) = 1$

$\rho_{c,sp,x} = \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right)\chi_{sp,x}\left(2 - t_{sp,x}\right) - t_{sp,x}\right] \frac{\gamma_{sp}}{\gamma_{M1}} & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\chi_{sp,x}\left(2 - t_{sp,x}\right) - t_{sp,x}\right]\gamma_{sp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\chi_{sp,x}\left(2 - t_{sp,x}\right) - t_{sp,x}\right]\gamma_{sp}}{\gamma_{M1}} > 1 \end{cases} = 1.028$

z-direction

$\psi_{sp,z} = 1$

$\alpha_{sp,sp} = 0.34$ $\lambda_{p0,sp} = 0.7$

$\phi_{sp,sp} = 0.5\left[1 + \alpha_{sp,sp}\left(\lambda_p - \lambda_{p0,sp}\right) + \lambda_p^2\right] = 0.897$

$\rho_{p,s,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{sp,sp} + \sqrt{\phi_{sp,sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.923$

Plate-like behaviour

$k_{\sigma,sp,sp,z} = 4$

$\sigma_{E,sp,z} = \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) b_{sp}^2} = 1.708\,MPa$

$\sigma_{cr,sp,sp,z} = k_{\sigma,sp,sp,z} \sigma_{E,sp,z} = 6.833\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,z} = \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) h_s^2} = 45.108\,MPa$

Interpolation

$t_{sp,z} = \min\left(\max\left(\frac{\sigma_{cr,sp,sp,z}}{\sigma_{cr,c,sp,z}} - 1.0\right); 1\right) = 0$

$\rho_{c,sp,z} = \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right)\chi_{sp,z}\left(2 - t_{sp,z}\right) - t_{sp,z}\right] \frac{\gamma_{sp}}{\gamma_{M1}} & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\chi_{sp,z}\left(2 - t_{sp,z}\right) - t_{sp,z}\right]\gamma_{sp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\chi_{sp,z}\left(2 - t_{sp,z}\right) - t_{sp,z}\right]\gamma_{sp}}{\gamma_{M1}} > 1 \end{cases} = 0.812$

Top plate stiffeners - check

x-direction

$\rho_{p,sp,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 0.981$

$\psi_{sp,x} = \frac{\sigma\left(h_w - t_{tp} - h_{sw}\right)}{\sigma\left(h_w - t_{tp}\right)} = 0.611$

$$\sigma_{E,cpt,x} := \frac{\pi^2 \cdot E \cdot t_w^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_w^2} = 2.479 \cdot 10^3 \ \text{MPa}$$

$$k_{\sigma,cpt,x} := \begin{cases} 1 & \text{if } \psi_{cpt,x} \geq 1 \\ \frac{8.2}{\left(1.05 + \psi_{cpt,x}\right)} & \text{if } 1 > \psi_{cpt,x} > 0 \\ 7.81 & \text{if } \psi_{cpt,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{cpt,x} + 9.78 \cdot \psi_{cpt,x}^2 & \text{if } 0 > \psi_{cpt,x} > -1 \\ 23.9 & \text{if } \psi_{cpt,x} = -1 \\ 5.98 \cdot \left(1 - \psi_{cpt,x}\right)^2 & \text{if } -1 > \psi_{cpt,x} > -3 \\ 95.68 & \text{if } \psi_{cpt,x} < -3 \end{cases} = 4.036$$

$$\sigma_{cr,cpt,pl,x} := k_{\sigma,cpt,x} \cdot \sigma_{E,cpt,x} = 1.001 \cdot 10^4 \ \text{MPa}$$

Column-like behaviour

$$\sigma_{E,c,cpt,x} := \frac{\pi^2 \cdot E \cdot i_w^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_w^2} = 12.147 \ \text{MPa}$$

$$\chi_{cpt} := \chi_0$$

Interpolation

$$\xi_{cpt,x} := \min\left(\max\left(\frac{\sigma_{cr,cpt,pl,x}}{\sigma_{E,c,cpt,x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c,cpt,x} := \begin{cases} \left[\left(p_{pl,cpt} - \chi_{cpt}\right) \cdot t_w \cdot \left(2 - \xi_{cpt,x}\right) - t_w^2\right] \cdot \frac{1}{A_{w1}} & \text{if } \frac{\left[\left(p_{pl,cpt} - \chi_{cpt}\right) \cdot t_w \cdot \left(2 - \xi_{cpt,x}\right) - t_w^2\right] \cdot m_{c0}}{A_{w1}} < 1 \\ 1 & \text{if } \frac{\left[\left(p_{pl,cpt} - \chi_{cpt}\right) \cdot t_w \cdot \left(2 - \xi_{cpt,x}\right) + \chi_{cpt}\right] \cdot m_{c0}}{A_{w1}} \geq 1 \end{cases} = 0.981$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,cpt}}{\frac{\rho_{c,cpt,x} \cdot f_y \cdot d}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,cpt}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.979$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,cpt}}{\frac{\rho_{c,cpt,x} \cdot f_y \cdot d}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,cpt}}{\frac{\rho_{c,cpt,x} \cdot f_y \cdot d}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,cpt}}{\frac{\rho_{c,cpt,x} \cdot f_y \cdot d}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z,Ed,cpt}}{\frac{\rho_{c,cpt,x} \cdot f_y \cdot d}{\gamma_{M1}}}\right) = 0.593$$
$$+ 3 \cdot \left(\frac{\tau_{Ed,cpt}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,cpt}}{\frac{\rho_{c,cpt,x} \cdot f_y \cdot d}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,cpt}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.414$$

EWM - Changed side plate thickness

This chapter covers the EWM when changing the side plate thickness to 8 mm

Geometry

Amount of:

Upper stiffeners: $n_{uo} := 9$

Lower stiffeners: $n_{lo} := 6$

Deck subpanels: $n_{s.so} := n_{uo} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350\,mm$

Side plate: $h_s := 389.2mm$ $t_s := 8mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su}\cdot\cos\left(21.12\cdot\frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{tx} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 10mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d\cdot b_d$ $A_s := 2t_s\cdot h_s$ $A_w := 2t_w\cdot h_w$ $A_b := t_b\cdot b_b$

$A_{so} := n_{so}\cdot t_{so}\cdot h_{so}$ $A_{su} := n_{su}\cdot t_{su}\cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.118\,m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of centroid of gravity, z_{tp}

$z_{tp} := \frac{\sum(A\cdot z)}{A_{tot}}$

(Formula to determine of gravity)

$$z_{tp} := \frac{A_d\frac{t_d}{2} + A_{so}\left(t_d + \frac{h_{so}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_b\left(t_d + h_s + h_{wy} + \frac{t_b}{2}\right) + A_{su}\left(t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right)}{A_{tot}} = 367.272\,mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\,mm$

Stiffeners over/under:

$$I_{y.st.so} := n_{so}\cdot\frac{t_{so}\cdot h_{so}^3}{12} + n_{so}\cdot t_{so}\cdot h_{so}\left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.697\times10^{-3}\,m^4$$

$I_{y.st.su.1} := 6\cdot1.506\cdot10^{-6}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2n_{su}\cdot h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 8.956\times10^{-4}\,m^4$$
$$+ 2n_{su}\cdot h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots$$
$$+ 2n_{su}\cdot h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2\cdot0.000312m^4$ from FEM

$$I_{y.w} := I_{y.w.1} + 2t_w\cdot h_w\left[h_w - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 3.027\times10^{-3}\,m^4$$

Deck:

$$I_{y.d} := \frac{b_d\cdot t_d^3}{12} + t_d\cdot b_d\left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.594\times10^{-7}\,m^4$$

Bottom flange:

$$I_{y.b} := \frac{b_b\cdot h_b^3}{12} + b_b\cdot h_b\left(h_b - z_{tp} - \frac{h_b}{2}\right)^2 = 4.239\times10^{-7}\,m^4$$

Side plate:

$$I_{y.s} := 2\cdot\frac{t_s\cdot h_s^3}{12} + 2t_s\cdot h_s\left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 2.434\times10^{-4}\,m^4$$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}}\cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}}\cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1-\psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$$k_{\sigma.4.2}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4\cdot\varepsilon(t)\cdot\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\cdot\max(3+\psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.089$

$$\text{Condition} := \begin{cases} 62\cdot\varepsilon\left(t_s\right)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42\cdot\varepsilon\left(t_s\right)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$$

$$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.45$

$s_{c.so} := s_{so} - t_{so}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.724$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.032\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 357.272\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s\left(h_{s.c}\right) = 357.272\,mm$

$h_{s.eff.c1} := 0.4\,h_{s.eff.c} = 142.909\,mm$

$h_{s.eff.c2} := 0.6\,h_{s.eff.c} = 214.363\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\,mm$

$A_{s.eff.c} := 2\,h_{s.eff.c}\cdot t_s = 5.716\times10^{-3}\,mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right)\cdot t_s = 6.227\times10^{-3}\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}\left(\psi_s\right)\frac{E\cdot\pi^2\cdot t_s^2}{12\left(1-\nu^2\right)h_s^2} = 677.64\,MPa$ EC1993-1-5 eq. A.1

Stiffeners

$h_{bv} := t_d = 10\,mm$

$h_{bv} := t_d + h_{so} = 150\,mm$

$\psi_{so} := \frac{\sigma_{sog}(z_2)}{\sigma_{sog}(z_1)} = 0.608$

$k_{\sigma.4.28}(\psi_{so}) = 0.61$ Higher stress close to support

Class4_so := "Class4" if $\frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon \cdot \sqrt{k_{\sigma.4.28}(\psi_{so})}$ = "Lower than class 4"
"Lower than class 4" otherwise

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.28}(\psi_{so}), b_{so}, t_{so}) = 0.485$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5:4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{-4} \cdot m^2$

Deck

$\frac{h_{so}}{t_d} = 35 \quad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ = "Class4" if $\frac{h_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4"
"Lower than class 4" otherwise

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} \cdot m$ $\psi_d := \psi_{sog}(z_d, z_d) = 1$

$k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$ [tab. 5.2 ES1993-1-1] $\psi_{sod} := \psi_{so} = 0.608$

$\lambda_{p.d} := \lambda_p(b_{\sigma.d}, \sigma_{x.so}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$b_{so.eff} := \rho_d \cdot b_{\sigma.so} = 321.461 \cdot mm$

Areas:

$A_{sog} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.31 \times 10^{-4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot n_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.053 \times 10^{-4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_d := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{-8} \cdot mm^4$

$\gamma := \frac{I_d}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 \cdot m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035 \cdot m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952 \quad \frac{1}{\gamma} \text{ if } \alpha \le \sqrt{\gamma} = 639.127$

$\lambda_{cp} := \begin{cases} \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} & \text{if } \sigma_{cr.p} \ne 0 \\ \infty & \text{otherwise} \end{cases} = 0.584$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ orthotropic Plate like behaviour

Effective area of the different panels/sl: $\frac{E \cdot \varepsilon^2}{\sigma_{cr.p}} = 9.886 \cdot MPa$ EC1993-1-5 eq.A.1

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d = 5.375 \times 10^{-4} \cdot mm^2$

Bucking strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^4 \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + n_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{d.c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{-7} \cdot mm^4$

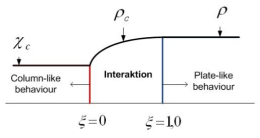
Bucking resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + n_{so} \cdot t_d = 5.74 \times 10^{-3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + n_{so.eff} \cdot t_d = 5.455 \times 10^{-3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.026 \times 10^3 \cdot MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$\bar{w} := \min\left(\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right]\right) = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{\bar{w}}{i}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{c.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{axst} := n_{so} \cdot n_{so.eff} \cdot t_d \cdot \sigma_{sog}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{sog}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.573 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{axst}}{n_{so} \cdot n_{so.eff} \cdot t_d + A_{so}} = -320.397 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{so})}{\gamma_M} = 281.032 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{so})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{so} \cdot t_d \cdot n_{so.eff} = 0.032 \cdot m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.115 \cdot m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) + \dots \\ &+ 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + \dots \right. \\ &\left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] + \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{sdy}) - \frac{h_{su.y}}{2}\right] + \dots \\ &+ A_b \cdot \left(h_{tb} - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 376.236 \cdot mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \dots \right. \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \dots \\ \left. + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 2.621 \times 10^{-4} \cdot m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot n_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot n_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.431 \times 10^{-3} \cdot m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015 \cdot m^4$

Negative moment

Effective cross-section with negative moment(lower flange in tension)

Area

$A_{td.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^{-4} \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_d) \cdot A_d}{\sigma_{com.Ed} \cdot \gamma_M} = 5.035 \times 10^{-3} \cdot mm^2$ (A.3)

$A_{td.eff.neg} := A_b$ Bottom flange in tension

A_s No reduction if the side plate is in class 4.

The effective area in compression is calculated according to 4.5.1(4)5. If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

A_{so} Stiffener class 4 --> no reduction

$A_{comp} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + n_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ A_{c.eff.loc} + n_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 432.173 \cdot cm^2$

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.062 \times 10^{-3} \cdot m^2$

A_b Tension -> No reduction

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.916 \cdot mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \left(t_{d.eff} + \frac{h_{so}}{2}\right) + n_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + \dots \\ &+ n_{so} \cdot n_{so.eff} \cdot t_d \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) + \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{sdy}) - \frac{h_{su.y}}{2}\right] + \dots \\ &+ A_b \cdot \left(h_{tb} - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 398.979 \cdot mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot n_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot n_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.99 \times 10^{-3} \cdot m^4$

$\bar{w} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot h_{sdy}) = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.084 \times 10^{-3} \cdot m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} - \dots = 7.438 \times 10^{-4} \cdot m^4$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{2}(h_{wy} - s_{sdy}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 - \dots$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{sdy}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 - \dots$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{sdy}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 \cdot m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[t_{tb} - \frac{h_b}{2} - \frac{1}{2}(h_{wy}) - z_{tp.eff.neg}\right]^2 = 2.502 \times 10^{-3} \cdot m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot h_b^3}{12} + t_b \cdot b_b \cdot \left(t_{tb} - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 3.733 \times 10^{-3} \cdot m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \dots \right. \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \dots \\ \left. + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 3.139 \times 10^{-4} \cdot m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.437 \times 10^{-10} \cdot mm^4$

Load

Load capacity:

$q_{_M} := 90 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is $q_{_M}$ when checking the moment capacity.

$Q_{_M} := q_{_M} \cdot b_d = 315 \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.276 \times 10^4\ kN \cdot m$

$q_V := 89 \frac{kN}{m^2}$ The load capacity is q_v when checking the shear capacity.

$Q_V := q_V \cdot b_d = 311.5 \frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 2.804 \times 10^3\ kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{sp.eff.neg}} = 0.036\ m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.278 \times 10^4\ kN \cdot m$

Moment capacity

$\frac{M_{Ed}}{M_{Rd}} = 0.998$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot t_{st}(t_w) \cdot t_w = 0.293\ m$

$\alpha_w := \min\left(\frac{h_{st}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_b}{2} + s_{stiff} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{st}}{2} \cdot \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right)}{2} \cdot h_{st} = 0.503\ m$

$a_{st.z.2} := a_{st.z.1} + s_{st} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\ m$

$a_{st.z.3} := a_{st.z.2} + s_{st} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\ m$

Moment of inertia stiffeners z-direction

$I_{st.z} := 2 \cdot t_{st} \cdot h_{st} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\ m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$\bar{z}_{web.z} := \frac{h_b}{2} + s_{stiff} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{st}}{2} = 0.525\ m$

$\bar{z}_{web.z.1} := a_{st.z.1} + s_{st} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\ m$

$\bar{z}_{web.z.2} := a_{st.z.2} + s_{st} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\ m$

Moment of inertia of 15st of the web z-direction

$I_{web.z} := 2 b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\ m^4$

$I_{st.z.tot} := I_{st.z} + I_{web.z} = 0.029\ m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \left| 5.34 + 4.00 \left(\frac{h_{s.eff}}{a}\right)^2 \text{ if } \frac{a}{h_{s.eff}} \geq 1 \right. \left. = 5.491 \right.$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\alpha_{sw} = 0$

$k_{\tau 2} := \max\left[\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\left[\min\left(t_f, t_w\right)^3 \cdot h_w\right]}}, \frac{2.1}{\min\left(t_f, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 1.191 \times 10^4$ EN 1993-1-5 (A.5)

SS-EN 1993-1-5 (5) ANM. (2) ger

$\chi_w := \left| \frac{\eta}{37.4 \cdot t_w \cdot \varepsilon\left(t_w\right) \cdot \sqrt{k_{\tau 1}}} \text{ if } \lambda_w < 0.83 \cdot \sqrt{\frac{k_{\tau 1}}{\eta}} \right. \left. = 1.2 \right.$ EC1-3-5 eq.5.6

$k_\tau := \left| 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau 1} \text{ if } \alpha \geq 3 \right. \left. = 5.391 \times 10^3 \right.$

$\left| 6.3 + 0.18 \frac{I_{st.z.tot}}{\min\left(t_f, t_w\right)^3 \cdot h_{web}} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_f, t_w\right)^3 \cdot h_{web}}} \right.$

$\left. 4.1 + \frac{I_{st.z.tot}}{\alpha^2} \right.$ otherwise

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\frac{\gamma_M}{\gamma_{M0}} = 1.2$ EC1993-1-3 5 ANM.2 5.1

$\text{Check} := \left| \text{"no shear buckling"} \text{ if } \frac{h_{web}}{\min\left(t_f, t_w\right)} \leq 31 \cdot \frac{\varepsilon\left(t_f\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \right. \left. = \text{"Reduce"} \right.$

$\left. \text{"Reduce"} \text{ otherwise} \right.$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon\left(t_w\right) \cdot \sqrt{k_\tau}} \cdot \lambda_{w1}\right) = 0.682$ Enligt SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \left| \eta \text{ if } \lambda_w < \frac{0.83}{\eta} \right. \left. = 1.2 \right.$ SS-EN 1993-1-5 5.3 Table 5.1, stiff end stiffener.

$\left| \frac{0.83}{\lambda_w} \text{ if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \right.$

$\left. \frac{1.37}{0.7 + \lambda_w} \text{ if } \lambda_w \geq 1.08 \right.$

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M1} := 1.00$ s.583 bärande konstruktioner

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_f) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 1.417 \times 10^3\ kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.804 \times 10^3\ kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.989$ (Contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_f) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4, 583 bärande konstruktioner

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{sf.eff.neg} := z_{sp.eff.neg} - \frac{t_d}{2} = 393.979\ mm$

$z_{sf.eff.neg} := h_b - \frac{h_b}{2} - z_{sp.eff.neg} = 483.021\ mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{sf.eff.neg} \cdot \left|z_{sf.eff.neg}\right| \cdot f_y(t_d)}{\gamma_{M0}} \ldots = 7.24 \times 10^3\ kN \cdot m$

$+ \frac{A_{sf.eff.neg} \cdot \left|z_{sf.eff.neg}\right| \cdot f_y(t_b)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{td} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_b \cdot \left(h_b + h_{web}\right)^2 \cdot f_y(t_b)}\right] = 0.684\ m$

$c_{td} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_b \cdot \left(h_b + h_{web}\right)^2 \cdot f_y(t_b)}\right] = 0.635\ m$

$V_{bf.Rd.cored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{td} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{td} \cdot \gamma_{M1} \cdot \gamma_n} = 270.678\ kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \left| V_{bf.Rd.cored} \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] \text{ if } M_{Ed} < M_{f.Rd.neg} \right. \left. = 0\ kN \right.$

$\left. 0 \text{ otherwise} \right.$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_f) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 1.417 \times 10^3\ kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.989$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.723 \times 10^{-3}\ m^4$

The flanges effective area:

$A_{eff.f.tot} := t_{sn.eff} \cdot t_d + n_{sn} \cdot t_{sn.eff} \cdot t_{d.eff} + n_{sn} \cdot t_{sn} \cdot t_{d.eff} + A_b = 432.579\ cm^2$

z_{fp} for the flanges effective areas:

$z_{tp.f.eff.neg} := \frac{\begin{matrix} t_{sn.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{sn} \cdot t_{sn.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \ldots \\ + n_{sn} \cdot t_{sn} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_b - \frac{h_b}{2}\right) \end{matrix}}{A_{eff.f.tot}} = 328.801\ mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.027\ m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 9.418 \times 10^3\ kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.278 \times 10^4\ kN \cdot m$

Moment capacity check when $q_M = 90 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.998$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 89 \frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.989$

The Reduced stress method

-Changed side plate thickness to 8 mm

Material

$f_{y0} := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ 335\,MPa & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$E = 210\,GPa$

$\nu = 0.3$ $G = \frac{E}{2\,(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$t_b = 16\,mm$ $b_b = 1000\,mm$ $A_b = b_b t_b = 0.016\,m^2$

Top plate

$t_d = 10\,mm$ $b_d = 3500\,mm$ $A_d = b_d t_d = 0.035\,m^2$

Side plates

$t_s = 8\,mm$ $h_s = 389.2\,mm$ $A_s = h_s t_s = 3.114 \cdot 10^{-3}\,m^2$

Webs

$t_w = 12\,mm$ $h_w = 1340\,mm$ $A_w = h_w t_w = 0.016\,m^2$

$h_{w,n} = 1250\,mm$ $h_{w,p} = 482.8\,mm$

Top stiffeners

$t_{st} = 16\,mm$ $h_{st} = 140\,mm$ $A_{st} = h_{st} t_{st} = 2.24 \cdot 10^{-3}\,m^2$

$s_{st} = 350\,mm$ $n_{st} = 9$

Web stiffeners

$t_{sw} = 12\,mm$ $h_{sw} = 120\,mm$ $A_{sw} = h_{sw} t_{sw} = 1.44 \cdot 10^{-3}\,m^2$

$s_{sw} = 440\,mm$ $n_{sw} = 3$

Whole bridge

$t_{bridge} = 18\,in$ $h_b = \frac{h_s}{2} + h_{w,p} + h_s + t_d = 0.89\,m$ $n_b = 2\,in$

$I_{yy} = 0.814676\,m^4$ From grasshopper

$V_{cg} = 0.522694\,m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q = 75\,\frac{kN}{m}$

$Q = q\,t_b = 262.5\,\frac{kN}{m}$

$M_{Ed} = \frac{Q\,t_{bridge}^2}{8} = 1.063 \cdot 10^4\,kN\,m$

$V_{Ed} = \frac{Q\,t_{bridge}}{2} = 2.362 \cdot 10^3\,kN$

Top plate

$\sigma_{x,Ed,top} = \frac{M_{Ed}}{I_{yy}}\left(h_w - \frac{t_d}{2} - V_{cg}\right) = 262.453\,MPa$ $\sigma_{z,Ed,top} = 0$

$S_{top} = \frac{t_d\,b_d}{n_{st}+2}\left(h_w - \frac{t_d}{2} - V_{cg}\right) = 1.153 \cdot 10^{-3}\,m^3$

$\tau_{Ed,top} = \frac{V_{Ed}\,S_{top}}{I_{yy}\,\min(t_s;t_{sw})} = 23.197\,MPa$

Side plates

$\sigma_{x,Ed,sg} = \frac{M_{Ed}}{I_{yy}}\left(h_w - t_d - V_{cg}\right) = 258.831\,MPa$

$\sigma_{z,Ed,sg} = \frac{Q}{n_b} = 16.406\,MPa$

$S_{sg} = \left(h_w - t_d - V_{cg}\right)t_s\frac{\left(h_w - t_d - V_{cg}\right)}{2} + \frac{t_d\,b_d}{n_{st}+2}\left(h_w - \frac{t_d}{2} - V_{cg}\right) = 9.747 \cdot 10^{-3}\,m^3$

$\tau_{Ed,sg} = \frac{V_{Ed}\,S_{sg}}{I_{yy}\,t_s} = 14.9\,MPa$

Governing slenderness from FE-software

$\sigma_{x,UNIT} = 4.68\,\frac{MPa}{\frac{kN}{m^2}}$ 90.066 MPa From a linear stress analysis in Sofistik

Top plate stiffeners

$\sigma_y = \sigma_{x,UNIT}\,q = 351\,MPa$

$\sigma_{x,Ed,top} = \frac{M_{Ed}}{I_{yy}}\left(h_w - t_d - V_{cg}\right) = 258.831\,MPa$

$\alpha_{ult} = \frac{\frac{t_d\,b_d}{n_{st}+2}}{\frac{t_d}{2} - \frac{t_d\,b_d}{n_{st}+2}} = 1.811$

$c = \frac{\frac{t_d}{2} - \frac{t_d\,b_d}{n_{st}+2}}{\left(h_w - t_d - \frac{h_{st}}{2} - V_{cg}\right) + \frac{t_d\,b_d}{n_{st}+2}\left(h_w - \frac{t_d}{2} - V_{cg}\right)} = 1.481 \cdot 10^{-3}\,m$

$\sigma_{x,UNIT} = 202.539\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} = \frac{\sigma_{x,UNIT}}{\sigma_y} = 2.701$

$\lambda_p = \sqrt{\frac{f_{y0}}{\sigma_{cr}}} = 0.612$ Slenderness for all parts

Top plate check

$\psi_{sg} = 1$

$\rho_{p,sg} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\,(3+\psi_{sg})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E,sg} = \frac{\pi^2\,E\,t_d^2}{12\,(1-\nu^2)\,b_d^2} = 1.549\,MPa$

$I_{yy,sg} = \frac{b_d\,t_d^3}{12\,(1-\nu^2)} = 3.205 \cdot 10^{-7}\,m^4$

$z_{sg,sg} = \frac{A_d\frac{t_d}{2} + n_{st}\,h_{st}\left(t_d + \frac{h_{st}}{2}\right)}{A_d + n_{st}\,h_{st}} = 32.411\,mm$

$I_{sg,sg} = \frac{b_d\,t_d^3}{12} + A_d\left(z_{sg,sg} - \frac{t_d}{2}\right)^2 + n_{st}\left[\frac{t_{st}\,h_{st}^3}{12} + A_{st}\left(t_d + \frac{h_{st}}{2} - z_{sg,sg}\right)^2\right] = 1.052 \cdot 10^{-6}\,mm^4$

$\gamma_{sg} = \frac{I_{sg,sg}}{z_{sg,sg}} = 328.143$

$k_{\sigma_g} = \frac{n_{st}\,A_{st}}{A_d} = 0.576$

$\alpha_{sg} = \min\left(\frac{z_{sg}}{b_d};0.5\right) = 0.571$

$k_{\sigma,p,sg} = \begin{cases} \frac{2\left[\left(1+\alpha_{sg}^2\right)^2 + \gamma_{sg} - 1\right]}{\alpha_{sg}^2\,(\alpha_{sg}+1)\,(1+k_{\sigma_g})} & \text{if } \alpha_{sg} \cdot \sqrt{\gamma_{sg}} < \sqrt{\gamma_{sg}} \\ \frac{4\,(1+\sqrt{\gamma_{sg}})}{(\alpha_{sg}+1)\,(1+k_{\sigma_g})} & \text{if } \alpha_{sg} \cdot \sqrt{\gamma_{sg}} \ge \sqrt{\gamma_{sg}} \end{cases} = 639.127$

$\sigma_{cr,p,sg} = k_{\sigma,p,sg}\,\sigma_{E,sg} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1,eff,sg} = \frac{3-\psi_{sg}}{5-\psi_{sg}}\,b_{sg} = 0.175\,m$

$b_{2,eff,sg} = \frac{2}{5-\psi_{sg}}\,b_{sg} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{c,t,sg} = b_{1,eff,sg} + b_{2,eff,sg} = 0.35\,m$

$A_{sg,t,sg} = t_d\,b_{c,t,sg} + A_{st} = 5.74 \cdot 10^{-3}\,m^2$

$z_{sg,d,t,sg} = \frac{t_d\,t_d\,b_{c,t,sg}\frac{t_d}{2} + A_{st}\left(t_d + \frac{h_{st}}{2}\right)}{t_d\,b_{c,t,sg} + A_{st}} = 0.034\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{t,t,sg} = \frac{b_{c,t,sg}\,t_d^3}{12} + b_{c,t,sg}\,t_d\left(z_{sg,d,t,sg} - \frac{t_d}{2}\right)^2 + \frac{t_{st}\,h_{st}^3}{12} + A_{st}\left(t_d + \frac{h_{st}}{2} - z_{sg,d,t,sg}\right)^2$

Combined moment of inertia

$\sigma_{cr,d,sg} = \frac{\pi^2\,E\,I_{t,t,sg}}{A_{sg,t,sg}\,z_{sg}^2} = 1.026 \cdot 10^3\,MPa$ Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,p} = \sigma_{cr,d}$

$\sigma_{cr,c,sg} = \sigma_{cr,d,sg}$

Reduction factor for column-like behaviour

$\lambda_p = \sqrt{\frac{f_{y0}}{\sigma_{cr,d,sg}}} = 0.045$

$e_{sg} = \max\left(e_{sg,d,t,sg} - z_{sg},\ t_d + \frac{h_{st}}{2} - z_{sg,d,t,sg}\right) = 0.046\,m$

$\alpha_{e,sg} = 0.49 + \frac{0.09}{\frac{2\,e_{sg}}{\gamma_{sg}}} = 0.582$ Stiffener with open cross section (curve c)

$\phi_{sg} = 0.5\left[1 + \alpha_{e,sg}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.807$ [6.3.1.2]

$\chi_{sg} = \min\left(\frac{1}{\phi_{sg} + \sqrt{\phi_{sg}^2 - \lambda_p^2}};1\right) = 0.75$

Interpolation

$t_{\psi,sg} = \min\left(\max\left(\frac{\sigma_{cr,p,sg}}{\sigma_{cr,c,sg}} - 1.0\right);1\right) = 0$

$\rho_{c,sg} = \begin{cases} \left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right] & \text{if } \frac{\left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right]\gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right]\gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.75$

Shear

$\eta = 1.2$ 7.1 (2)

$\chi_w = \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} < \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p > 1.08 \end{cases} = 1.2$

$\frac{\sigma_{cr,sg}\,\alpha_{ult}}{\gamma_{M0}} = 0.758$

Side plates check

x-direction

$\sigma_{max} = f_y(t_s)$

$\sigma_{t0} = \left[\frac{-\sigma_{min}}{f_y(t_s)} + 1 + \sigma_{min}\right] - 1$

$\psi_{sg,x} = \frac{\sigma\left(h_w - t_d - h_s\right)}{\sigma\left(h_w - t_d\right)} = -0.089$

$\sigma_{x,sg} = \frac{M_{Ed}}{I_{yy}}\left(h_w - t_d - V_{cg}\right) = 258.831\,MPa$

$\sigma_{x,bot} = \frac{M_{Ed}}{I_{yy}}\left(h_w - t_d - h_s - V_{cg}\right) = -23.104\,MPa$

$\psi_{sg1,x} = \max\left(3 + \psi_{sg,x};0\right) = 2.911$ $\psi_{sg1} = \frac{\sigma_{x,bot}}{\sigma_{x,sg}} = -0.089$

$\rho_{p,sg,x} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\,(\psi_{sg1,x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$\sigma_{E,sg,x} = \frac{\pi^2\,E\,t_s^2}{12\,(1-\nu^2)\,b_s^2} = 80.192\,MPa$

$k_{\sigma,sg,x} = \begin{cases} 1 & \text{if } \psi_{sg,x} \ge 1 \\ \frac{8.2}{1.05 + \psi_{sg,x}} & \text{if } 1 > \psi_{sg,x} > 0 \\ 7.81 & \text{if } \psi_{sg,x} = 0 \\ 7.81 - 6.29\,\psi_{sg,x} + 9.78\,\psi_{sg,x}^2 & \text{if } 0 > \psi_{sg,x} > -1 \\ 23.9 & \text{if } \psi_{sg,x} = -1 \\ 5.98\left(1 - \psi_{sg,x}\right)^2 & \text{if } -1 > \psi_{sg,x} > -3 \\ 95.68 & \text{if } \psi_{sg,x} < -3 \end{cases} = 8.449$

Column-like behaviour

$\sigma_{cr,sg,x} = \frac{\pi^2\,E\,t_s^2}{12\,(1-\nu^2)\,b_s^2} = 3.037\,MPa$

$\alpha_{sg} = 0.21$

$\phi_{sg} = 0.5\left[1 + \alpha_{sg}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.731$

$\chi_{sg} = \min\left(\frac{1}{\phi_{sg} + \sqrt{\phi_{sg}^2 - \lambda_p^2}};1\right) = 0.885$

Interpolation

$t_{\psi,sg,x} = \min\left(\max\left(\frac{\sigma_{cr,p,sg,x}}{\sigma_{cr,c,sg,x}} - 1.0\right);1\right) = 1$

$\rho_{c,sg,x} = \begin{cases} \left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg,x}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right] & \text{if } \frac{\left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg,x}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right]\gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg,x}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right]\gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

Z-direction

$\psi_{sg,z} = 1$

$\alpha_{p,sg} = 0.34$ $\chi_{p0,sg} = 0.7$

$\phi_{p,sg} = 0.5\left[1 + \alpha_{p,sg}\left(\lambda_p - \chi_{p0,sg}\right) + \lambda_p^2\right] = 0.791$

$\rho_{p,sg,z} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{p,sg} + \sqrt{\phi_{p,sg}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,p,sg,z} = 4$

$\sigma_{E,sg,z} = \frac{\pi^2\,E\,t_{sw}^2}{12\,(1-\nu^2)\,b_s^2} = 3.037\,MPa$

$\sigma_{cr,p,sg,z} = k_{\sigma,p,sg,z}\,\sigma_{E,sg,z} = 12.147\,MPa$

Column-like behaviour

$\sigma_{cr,c,sg,z} = \frac{\pi^2\,E\,t_{sw}^2}{12\,(1-\nu^2)\,b_s^2} = 80.192\,MPa$

Interpolation

$t_{\psi,sg,z} = \min\left(\max\left(\frac{\sigma_{cr,p,sg,z}}{\sigma_{cr,c,sg,z}} - 1.0\right);1\right) = 0$

$\rho_{c,sg,z} = \begin{cases} \left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg,z}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right] & \text{if } \frac{\left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg,z}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right]\gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sg} - \chi_{sg}\right)k_{\psi,sg,z}\left(2 - t_{\psi,sg}\right) + \chi_{sg}\right]\gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.885$

Top plate stiffeners - check

x-direction

$\rho_{p,sg,x} = \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$\psi_{sg,x} = \frac{\sigma\left(h_w - t_d - h_s\right)}{\sigma\left(h_w - t_d\right)} = 0.608$

$\sigma_{E,sg,x} = \frac{\pi^2\,E\,t_{sg}^2}{12\,(1-\nu^2)\,b_d^2} = 2.479 \cdot 10^3\,MPa$

$k_{\sigma,sg,x} = \begin{cases} 1 & \text{if } \psi_{sg,x} = 1 \\ \frac{8.2}{1.05 + \psi_{sg,x}} & \text{if } 1 > \psi_{sg,x} > 0 \\ 7.81 & \text{if } \psi_{sg,x} = 0 \\ 7.81 - 6.29\,\psi_{sg,x} + 9.78\,\psi_{sg,x}^2 & \text{if } 0 > \psi_{sg,x} > -1 \\ 23.9 & \text{if } \psi_{sg,x} = -1 \\ 5.98\left(1 - \psi_{sg,x}\right)^2 & \text{if } -1 > \psi_{sg,x} > -3 \\ 95.68 & \text{if } \psi_{sg,x} < -3 \end{cases} = 4.062$

$\sigma_{cr,c,sp,s,x} := k_{\sigma,sp,s,x} \cdot \sigma_{E,sp,s,x} = 1.007 \times 10^{4}\ \text{MPa}$

Column-like behaviour

$\sigma_{cr,c,sp,s,x} := \frac{\pi^2 \cdot E_{sp}^2}{12 \cdot (1 - \nu^2)^2 \cdot \lambda_{sp}^2} = 12.147\ \text{MPa}$

$\lambda_{sp,s,x} := \lambda_{sp}$

Interpolation

$\xi_{sp,s,x} := \min\left(\max\left(\frac{\sigma_{cr,c,sp,s,x}}{\sigma_{cr,c,sp,s,x}} - 1.0\right), 1\right) = 1$

$\rho_{c,sp,s,x} := \begin{cases} \left[\left(\rho_{sp,sp,s,x} - \chi_{sp}\right)\cdot k_{sp,s,x}\cdot\left(2 - \xi_{sp,s,x}\right) - \dots\right] \\ + \chi_{sp} \end{cases} \text{if } \frac{\left[\left(\rho_{sp,sp,s,x} - \chi_{sp}\right)\cdot k_{sp,s,x}\cdot\left(2 - \xi_{sp,s,x}\right) + \chi_{sp}\right]\cdot\sigma_{sx}}{\gamma_{M1}} < 1 \quad = 1$

$\text{if } \frac{\left[\left(\rho_{sp,sp,s,x} - \chi_{sp}\right)\cdot k_{sp,s,x}\cdot\left(2 - \xi_{sp,s,x}\right) + \chi_{sp}\right]\cdot\sigma_{sx}}{\gamma_{M1}} = 1$

Top plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,s}\cdot f_y\cdot t_{sp}}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w\cdot f_y\cdot t_{sp}}{\gamma_{M1}}}\right)^2 = 0.981$

Side plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,s}\cdot f_y\cdot t_{sp}}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,s}\cdot f_y\cdot t_{sp}}{\gamma_{M1}}}\right)^2 \cdot \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,s}\cdot f_y\cdot t_{sp}}{\gamma_{M1}}}\right) \cdots = 0.655$

$+ 3\cdot\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w\cdot f_y\cdot t_{sp}}{\gamma_{M1}}}\right)^2$

Top plate stiffeners check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,s}\cdot f_y\cdot t_{sp}}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w\cdot f_y\cdot t_{sp}}{\gamma_{M1}}}\right)^2 = 0.535$

EWM - Changed side plate thickness

This chapter covers the EWM when changing the side plate thickness to 10 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s.sp} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{su} := \frac{b_d}{n_{s.su}} = 350 \cdot mm$

Side plate: $h_s := 389.2mm$ $t_s := 10mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140mm$ $t_{su} := 16mm$ $a_{su} := 6mm$

Stiffeners under: $h_{sl} := 120mm$ $t_{sl} := 12mm$ $a_{sl} := 5mm$

$h_{sl.y} := h_{sl} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{tx} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 864 \cdot mm$

Poissons ratio $\upsilon := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d \cdot b_d$ **Parameter:** $A_s := 2 \cdot t_s \cdot h_s$ $A_w := 2 \cdot t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{su} := n_{su} \cdot t_{su} \cdot b_{su}$ $A_{sl} := n_{sl} \cdot t_{sl} \cdot b_{sl}$

$E := 210GPa$

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.12 \cdot m^2$ -1 tab. 3.1

$A_{-} := A_{su} + A_d = 0.055 \cdot m^2$ ##

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Change of center of gravity, z_{sp}

(From top to centre of gravity)

$$z_{sp} := \frac{A_d \cdot \frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b\left(t_d + h_s + h_{wy}\right) + A_{sl}\left[t_d + h_s + \frac{1}{2}\left(h_{wy} - s_{su0}\right) - \frac{h_{sl.y}}{2}\right]}{A_{tot}} = 365.157 \cdot mm$$

Moment of inertia I_y

$s_{su0.y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.su} := n_{su} \cdot \left[\frac{t_{su} \cdot h_{su}^3}{12} + A_{su} \cdot t_{su} \cdot h_{su} \cdot \left(t_d + \frac{h_{su}}{2} - z_{sp}\right)^2\right] = 1.672 \times 10^{-3} \cdot m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-5} \cdot m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{2}\left(h_{wy} - s_{su0.y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \ldots = 9.064 \times 10^{-4} \cdot m^4 + 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0.y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \ldots + 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0.y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.00312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 \cdot t_w \cdot h_w \cdot \left[h_b - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{sp}\right]^2 = 3.064 \times 10^{-3} \cdot m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{sp}\right)^2 = 4.54 \times 10^{-3} \cdot m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_b - z_{sp} - \frac{h_b}{2}\right)^2 = 4.274 \times 10^{-7} \cdot m^4$

Side plate:

$I_{y.s} := 2 \cdot \frac{t_s \cdot h_s^3}{12} + 2 \cdot t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{sp}\right)^2 = 2.989 \times 10^{-4} \cdot m^4$

Total:

$I_{y.tot} := I_{y.st.su} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015 \cdot m^4$

Reduced cross section

Functions to investigate the stresses

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{sp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{sp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2j}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.096$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.503$

$s_{c.ss} := s_{s0} - s_{s0}$ [table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.578$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{sp} = 0.034 \cdot m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 355.157 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 355.157 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 142.063 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 213.094 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 7.103 \times 10^{-3} \cdot mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right) \cdot t_s = 7.784 \times 10^{-3} \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}\left(\psi_s\right) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot \left(1 - \upsilon^2\right) \cdot b_s^2} = 1.065 \times 10^3 \cdot MPa$ EC1993-1-5 eq. A.1

Stiffeners

$t_{su} := t_d = 10 \cdot mm$

$h_{sw} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.606$

$k_{\sigma.4.2b}(\psi_{so}) = 0.611$ Higher stress close to support

Class4_ := "Class4" if $\frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon_{so} \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})}$ = "Lower than class 4"
"Lower than class 4" otherwise

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2b}(\psi_{so}), b_{so}, t_{so}\right) = 0.484$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_d) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so} \cdot b_{so} = 140\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot b_{so.eff} = 2.016 \times 10^{-4}\ m^2$

Deck

$\frac{b_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ := "Class4" if $\frac{b_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4"
"Lower than class 4" otherwise

$z_{cd} := \frac{-t_d}{2} = -5 \times 10^{-3}\ m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$b_{sw} := b_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, b_{sw}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$b_{so.eff} := \rho_d \cdot b_{so} = 321.461\ mm$

Areas:

$A_{sw} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.31 \times 10^{-4}\ m^2$

$A_{c.eff.loc} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.053 \times 10^{-4}\ m^2$

More than three longitudinal stiffeners --> $k_{\sigma p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5}\ m^4$

$y_{tp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411\ mm$

$I_d := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{-8}\ m^4$

$\gamma := \frac{I_d}{I_p} = 328.143 \qquad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02\ m^2$ EC1993-1-5 A.1 (2)

Critical buckling stress, orthotropic plate:

$A_{\alpha} := b_d \cdot t_d = 0.03\ m^2$

$\sigma_{cr.p} := k_{\sigma p}(\gamma, \alpha, \psi) \cdot \frac{E \cdot \pi^2 \cdot t_d^2}{12 \cdot (1 - \nu^2) \cdot b_d^2} = 990.256\ MPa$ EC1993-1-5 eq.A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952 \qquad \bar{\sigma}_{cr.p} := \left(\frac{\sqrt{\beta_{A.c}} + 1}{2}\right)^2 \text{ if } \alpha \le \sqrt{\gamma} \quad = 639.127$

$\lambda_{pw} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\bar{\sigma}_{cr.p}}} = 0.584$ otherwise

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d = 5.375 \times 10^{-4}\ m^2$

Bucking strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^{3}\ kN$

Column like behaviour

$y_{tp.sl.c} := \frac{n_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + n_{so} \cdot t_d} = 34.268\ mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_{so}^3}{12} + n_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{-7}\ m^4$

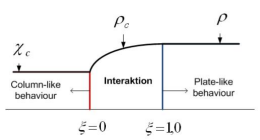
Bucking resistance stiffener

$A_{sw} := t_{so} \cdot h_{so} + n_{so} \cdot t_d = 5.74 \times 10^{-3}\ m^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + n_{so.eff} \cdot t_d = 5.455 \times 10^{-3}\ m^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot z^2} = 1.026 \times 10^{3}\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sw})}{\sigma_{cr.sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\ mm$

$z_c := \min\left(\left|\left(y_{tp.sl} - \frac{t_d}{2}\right)\right|, \left|\left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right|\right) = 47.589\ mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\left(\frac{\sigma_{c.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero-> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(z_d, \frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(z_d + \frac{h_{so.eff}}{2}\right) = -1.572 \times 10^{4}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -320.196\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_d)}{\gamma_M} = 281.032\ MPa$

$K_{A.2.1} :=$ "OK" if $\frac{\rho_c \cdot f_y(t_d)}{\gamma_M} > \sigma_{com.Ed}$ = "OK"
"EJ OK" otherwise

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{so} \cdot t_d \cdot s_{so.eff} = 0.032\ m^2$

$A_{tot.eff} := A_{sw.eff} + A_{d.eff} + A_w + A_{fst} + A_b + A_{s.eff} = 0.117\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{sw.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) + \dots \\ &+ 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + \dots \right. \\ &\left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] + \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wt}}{2}\right) + A_{fst} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{stb}) - \frac{h_{ws.t}}{2}\right] + \dots \\ &+ A_b \cdot \left(h_n - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 373.95\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \dots \right. \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \dots \\ \left. + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 3.215 \times 10^{-4}\ m^4$

$I_{y.d.eff} := \frac{n_{so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{so} \cdot s_{so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.376 \times 10^{-7}\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\ m^4$

Negative moment

Effective cross-section with negative moment(lower flange in tension)

Area

$A_{bf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^{-4}\ m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_d) \cdot A_d}{\sigma_{com.Ed} \cdot \gamma_M} = -5.038 \times 10^{-3}\ m^2$ (A.3)

$A_{bf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{w.eff} + A_w + A_{fst} + A_b = 1.078 \times 10^{-3}\ m^2 = 432.173\ cm^2$

Class 4 sections should be designed using the maximum stress in the compression part unless it is permissible to use reduced stresses in the tension part. Therefore the web-thickness is changed with another reduction factors.

$t_{w.eff} := \begin{cases} t_w \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \text{otherwise} \end{cases} = 7.916\ mm$

$z_{tp.eff} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \left(t_{d.eff} + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} - \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) - \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) - \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wt}}{2}\right) + A_{fst} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{stb}) - \frac{h_{ws.t}}{2}\right] - \dots \\ &+ A_b \cdot \left(h_n - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 394.694\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so} \cdot s_{so} \cdot t_{d.eff}^3}{12} + t_d \cdot n_{so} \cdot s_{so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.882 \times 10^{-3}\ m^4$

$h_{wt.eff} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{st0} = 7.206\ mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{s_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot s_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.029 \times 10^{-3}\ m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-7}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} + \dots \\ + 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{stby}) - \frac{h_{wx}}{2} - z_{tp.eff.neg}\right]^2 + \dots \\ + 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{stby}) - \frac{h_{wx}}{2} - z_{tp.eff.neg}\right]^2 + \dots \\ + 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{stby}) - \frac{h_{wx}}{2} - z_{tp.eff.neg}\right]^2 = 7.633 \times 10^{-4}\ m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 t_w \cdot h_w \cdot \left[h_n - \frac{h_b}{2} - \frac{1}{2}(h_{wy}) - z_{tp.eff.neg}\right]^2 = 2.569 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot h_b^3}{12} + t_b \cdot b_b \cdot \left(h_n - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 3.8 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \dots \right. \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \dots \\ \left. + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 3.795 \times 10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.442 \times 10^{-10}\ mm^4$

Load

Load capacity:

$q_M := 91 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_d = 318.5 \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.29 \times 10^{4}\ kN \cdot m$

$q_V := 112\frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V \cdot b_{bd} = 392\frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot b_{bridge}}{2} = 3.528 \times 10^3 kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.037\,m^3$

$M_{Rd} := W_{y.eff}\frac{f_d(t_d)}{\gamma_{M1}} = 1.297 \times 10^4 kN \cdot m$

Moment capacity

$\frac{M_{Ed}}{M_{Rd}} = 0.994$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeners

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot t_w = 0.293\,m$

$\alpha_w := \min\left(\frac{h_{wy}}{b_w}\right)\frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{wall}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) + \frac{t_{ws}}{2} - \frac{\sin\left(\alpha_w\frac{\pi}{180}\right)\cdot b_{ws}}{2} = 0.503\,m$

$a_{st.z.2} := a_{st.z.1} + s_{ws}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) = 0.913\,m$

$a_{st.z.3} := a_{st.z.2} + s_{ws}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) = 1.324\,m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 n_{ws} b_{ws}\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3} m^4$

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

~~$a_{web.z.1} := \frac{b_b}{2} + s_{wall}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) + \frac{t_{ws}}{2} = 0.525\,m$~~

~~$a_{web.z.2} := a_{st.z.1} + s_{ws}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) = 0.935\,m$~~

~~$a_{web.z.3} := a_{st.z.2} + s_{ws}\cdot\cos\left(\alpha_w\frac{\pi}{180}\right) = 1.346\,m$~~

Moment of inirtia of 15st of the web z-direction

$I_{ww.z} := 2 b_{web} t_w\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\,m^4$

$I_{st.z.tot} := I_{st.z} + I_{ww.z} = 0.029 m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00\left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34\left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{wi} = 0$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau s1} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 h_{web}} + 2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 h_{web}}} & \text{otherwise} \\ 4.1 + \frac{}{\alpha^2} \end{cases} = 2.789 \times 10^3$

Slenderness for the shear buckling check has to be performed for stiffened web:

$\lambda_{w1} := \frac{h_{web}}{37.4 t_w \varepsilon\sqrt{k_\tau}} = 0.546$ EC1993-1-5 5.3 ANM 2 5.1

$\gamma_M := 1.2$

Check := "no shear buckling" if $\frac{h_{web}}{\min\left(t_w, t_w\right)} \leq 31\frac{\varepsilon\left(t_d\right)}{\eta}\sqrt{k_{\tau 1}}$ = "Reduce"

"Reduce" otherwise

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \varepsilon\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.546$ Enligt SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ SS-EN 1993-1-5 5.3 Table 5.1, stiff end stiffener.

Kapacity of the web

~~$\gamma_{M0} := 1.00$~~ $\gamma_{M1} := 1.2$ Safety class 3

~~$\gamma_{M1} := 1.00$~~ s.SB3 bärande konstruktioner

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_d(t_d) \cdot h_{web} \cdot t_s}{\sqrt{3}\cdot\gamma_{M1}\cdot n} = 1.771 \times 10^3 kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 3.528 \times 10^3 kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.996$ (Contribution from two web)

$V_{b.Rd} \leq \frac{\eta \cdot f_d(t_d) \cdot h_{web} \cdot t_s}{\sqrt{3}\cdot\gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4, SB3 bärande konstruktioner

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{tf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 389.694\,mm$

$z_{uf.eff.neg} := h_n - \frac{t_b}{2} - z_{tp.eff.neg} = 487.306\,mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg}\left|z_{tf.eff.neg}\right| f_d(t_d)}{\gamma_{M0}} \ldots = 7.215 \times 10^3 kN \cdot m$
$+ \frac{A_{uf.eff.neg}\left|z_{uf.eff.neg}\right| f_d(t_b)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} \leq M_{f.Rd.neg} = 0$

$c_{tf} := z\left[0.25 + \frac{1.6\, b_d t_d z^2 f_d(t_d)}{t_s\left(h_n + b_{wy}\right)^2 f_d(t_s)}\right] = 0.647\,m$

$c_{uf} := z\left[0.25 + \frac{1.6\, b_b t_b z^2 f_d(t_b)}{t_s\left(h_n + b_{wy}\right)^2 f_d(t_s)}\right] = 0.608\,m$

$V_{bf.Rd.cored} := \frac{b_d \cdot t_d^2 \cdot f_d(t_d)}{c_{tf}\cdot\gamma_{M1}\cdot n} + \frac{b_b \cdot t_b^2 \cdot f_d(t_b)}{c_{uf}\cdot\gamma_{M1}\cdot n} = 284.577 kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cored}\left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} = 0\,kN \\ 0 & \text{otherwise} \end{cases}$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_d(t_d) \cdot h_{web} \cdot t_s}{\sqrt{3}\cdot\gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

~~$V_{w.Rd}$~~ $:= V_{bw.Rd} + V_{bf.Rd} = 1.771 \times 10^3 kN$

Check:

$\frac{V_{Ed}}{2} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.996$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.682 \times 10^{-3} m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff}\cdot t_d + n_{so}\cdot s_{so.eff}\cdot t_{d.eff} + n_{su}\cdot s_{su}\cdot t_{d.eff} + A_b = 432.579\,cm^2$

z_{tp} for the flanges effective areas:

$z_{tp.f.eff.neg} := \frac{s_{so.eff}\cdot t_d\left(\frac{t_d}{2}\right) + n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\left(\frac{t_{d.eff}}{2}\right) \ldots}{A_{eff.f.tot}} = 328.801\,mm$
$+ n_{su}\cdot s_{su}\cdot t_{d.eff}\left(\frac{t_{d.eff}}{2}\right) + A_b\left(h_n - \frac{t_b}{2}\right)$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.026\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff}\frac{f_d(t_d)}{\gamma_M} = 9.374 \times 10^3 kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.297 \times 10^4 kN \cdot m$

Moment capacity check when $q_M = 91\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.994$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 112\frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.996$

The Reduced stress method

-Changed side plate thickness to 10 mm

Material

$f_y(t) := \begin{cases} 355\,MPa & \text{if } t < 40\,mm \\ 335\,MPa & \text{if } t < 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$E := 210\,GPa$

$\nu := 0.3$ $G := \frac{E}{2\,(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate:

$t_b := 16mm$ $b_b := 1000mm$ $A_b := b_b t_b = 0.016m^2$

Top plate:

$t_p := 10mm$ $b_p := 3500mm$ $A_p := b_p t_p = 0.035m^2$

Side plates:

$t_s := 10mm$ $h_s := 389.2mm$ $A_s := h_s t_s = 3.892 \times 10^{-3}\,m^2$

Webs:

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w t_w = 0.016m^2$

$h_{w,k} := 1250mm$ $h_{w,fr} := 482.8mm$

Top stiffeners:

$t_{st} := 16mm$ $h_{st} := 140mm$ $A_{st} := h_{st} t_{st} = 2.24 \times 10^{-3}\,m^2$

$s_{st} := 350mm$ $n_{st} := 9$

Web stiffeners:

$t_{st} := 12mm$ $h_{st} := 120mm$ $A_{st} := h_{st} t_{st} = 1.44 \times 10^{-3}\,m^2$

$s_{st} := 440mm$ $n_{st} := 3$

Whole bridge:

$t_{bridge} := 18in$ $h_b := \frac{h_s}{2} + h_{w,fr} + h_p + t_d = 0.89m$ $s_b := 2m$

$I_{yd} := 0.01473\,m^4$ From grasshopper

$y_{fg} := 0.524853m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_{fk} := 77\,\frac{kN}{m}$

$Q := q_{fk} = 269.5\,\frac{kN}{m}$

$M_{Ed} := \frac{Q\,t_{bridge}^2}{8} = 1.091 \times 10^4\,kN\,m$

$V_{Ed} := \frac{Q\,t_{bridge}}{2} = 2.425 \times 10^3\,kN$

Top plate:

$\sigma_{x,Ed,tp} := \frac{M_{Ed}}{I_{yd}} \left(h_w - y_{fg} - \frac{t_p}{2} - y_{fg} \right) = 266.702\,MPa$ $\sigma_{z,Ed,tp} := 0$

$S_{tp} := \frac{t_p b_p}{n_{st}+2} \left(h_w - y_{fg} - \frac{t_p}{2} - y_{fg} \right) = 1.146 \times 10^{-3}\,m^3$

$\tau_{Ed,tp} := \frac{V_{Ed} S_{tp}}{I_{yd} \min(t_p, t_{st})} = 18.858\,MPa$

Side plates:

$\sigma_{x,Ed,sp} := \frac{M_{Ed}}{I_{yd}} \left(h_w - y_{fg} - y_{fg} \right) = 262.999\,MPa$

$\sigma_{z,Ed,sp} := \frac{0}{n_{st}} = 13.475\,MPa$

$S_{sp} := \left(h_w - y_{fg} - y_{fg} \right) t_s \frac{\left(h_w - y_{fg} - y_{fg} \right)}{2} + \frac{t_p b_p}{n_{st}+2} \left(h_w - \frac{t_p}{2} - y_{fg} \right) = 9.808 \times 10^{-3}\,m^3$

$S_{tps} := \left(\frac{\left(\frac{t_w h_w}{2} + 2\,\frac{t_p b_p}{n_{st}+2} \right)}{n_{st}+2} \left(h_w - y_{fg} - \frac{h_{st}}{4} - y_{fg} \right) \ldots + 2\,\frac{t_p b_p}{n_{st}+2} \left(h_w - y_{fg} - \frac{t_p}{2} - y_{fg} \right) \right) = 1.48 \times 10^{-3}\,m^3$

$\tau_{Ed,sp} := \frac{V_{Ed} S_{tps}}{I_{yd} t_s} = 15.223\,MPa$

$\sigma_{v,Ed,sp} := \sqrt{\sigma_{x,Ed,sp}^2 + \sigma_{z,Ed,sp}^2 - \sigma_{x,Ed,sp}\,\sigma_{z,Ed,sp} + 3\,\tau_{Ed,sp}^2} = 262.999\,MPa$

Governing slenderness from FE-software

$\sigma_{V,UNIF} := 4.93\,\frac{kN}{m^2}$ From a linear stress analysis in Solidsk

$\sigma_{cr} := \sigma_{V,UNIF}\,\eta = 348.81\,MPa$

$\alpha_{ult} := \frac{f_y(t_p)}{\sigma_{cr}} = 1.018$

$\alpha_{cr,UNIF} := 220.027\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Solidsk

$\eta_{cr} := \frac{\alpha_{cr,UNIF}}{\eta} = 2.868$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.596$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{p,tp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\,(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E,tp} := \frac{\pi^2\,E\,t_p^2}{12\,(1-\nu^2)\,b_p^2} = 1.549\,MPa$

$I_{yp,tp} := \frac{b_p t_p^3}{12\,(1-\nu^2)} = 3.205 \times 10^{-7}\,m^4$

$y_{sp,tp,sp} := \frac{A_p \frac{t_p}{2} + n_{st} A_{st} \left(t_p + \frac{h_{st}}{2} \right)}{A_p + n_{st} A_{st}} = 32.411\,mm$

$I_{y,tp} := \frac{b_p t_p^3}{12} + A_p \left(y_{sp,tp,sp} - \frac{t_p}{2} \right)^2 \ldots + n_{st} \left[\frac{t_{st} h_{st}^3}{12} + A_{st} \left(t_p + \frac{h_{st}}{2} - y_{sp,tp,sp} \right)^2 \right] = 1.052 \times 10^{-8}\,mm^4$

$\gamma_{tp} := \frac{I_{yd,tp}}{I_{y,tp}} = 328.143$

$\delta_{tp} := \frac{n_{st} A_{st}}{A_p} = 0.576$

$\alpha_{tp} := \min \left(\frac{h_s}{t_p}, 0.5 \right) = 0.571$

$k_{\sigma,p,tp} := \begin{cases} \frac{2 \left[\left(1+\alpha_{tp} \right)^2 + \gamma_{tp} - 1 \right]}{\alpha_{tp} \cdot (\alpha_{tp}-1)\,(1+\delta_{tp})} & \text{if } \alpha_{tp} \cdot \sqrt{\gamma_{tp}} < 639.127 \\ 4\,(1+\sqrt{\square}) & \text{if } \alpha_{tp} \cdot \sqrt{\gamma_{tp}} > 639.127 \end{cases}$

$\sigma_{cr,p,tp} := k_{\sigma,p,tp} \sigma_{E,tp} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1,tot,tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}} s_{st} = 0.175m$

$b_{2,sup,tp} := \frac{2}{5-\psi_{tp}} s_{st} = 0.175m$ Parts of the top plate that contributes to the stiffener

$A_{st,t,tp} := b_{1,tot,tp} \cdot t_p + b_{2,sup,tp} \cdot t_p = 0.35m$

$y_{sp,st,tp} := \frac{t_p t_{st} \frac{t_p}{2} + A_{st} \left(t_p + \frac{h_{st}}{2} \right)}{t_p t_{st} + A_{st}} = 0.034m$ combined center of gravity for stiffener and contributing top plate.

$I_{st,t,tp} := \frac{t_{st} h_{st}^3}{12} + t_{st} h_{st} \left(t_{sp,st,tp} - \frac{h_{st}}{2} \right)^2 + \frac{t_p b_{st}^3}{12} + A_{st} \left(t_p + \frac{h_{st}}{2} - y_{sp,st,tp} \right)^2$ Combined moment of inertia

$\sigma_{cr,st,tp} := \frac{\pi^2\,E\,I_{st,t,tp}}{A_{st,t,tp}\,s_{st}^2} = 1.026 \times 10^{-3}\,MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,p} = \sigma_{cr,st}$

$\sigma_{cr,c,tp} := \sigma_{cr,st,tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{st,t,tp}}{A_{st,t,tp}}} = 0.045m$

$e_{tp} := \max \left(y_{sp,st,tp} - y_{fg} + \frac{t_p}{2} - y_{sp,st,tp} \right) = 0.046m$

$\alpha_{e,tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5 \left[1 + \alpha_{e,tp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 0.793$ [6.3.1.2]

$\chi_{tp} := \min \left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1 \right) = 0.76$

Interpolation

$\xi_{tp} := \min \left(\max \left(\frac{\sigma_{cr,tp,sp}}{\sigma_{cr,c,tp}} - 1.0 \right), 1 \right) = 0$

$\rho_{c,tp} := \begin{cases} \left[\left(\rho_{p,tp,sp} - \chi_{tp} \right) k_{tp} \left(2 - \xi_{tp} \right) + \chi_{tp} \right] & \text{if } \frac{\left[\left(\rho_{p,tp,sp} - \chi_{tp} \right) k_{tp} \left(2 - \xi_{tp} \right) + \chi_{tp} \right] t_{st}}{t_{bridge}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp,sp} - \chi_{tp} \right) k_{tp} \left(2 - \xi_{tp} \right) + \chi_{tp} \right] t_{st}}{t_{bridge}} > 1 \end{cases} = 0.76$

Shear

$\eta = 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} < \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p > 1.08 \end{cases} = 1.2$

$\frac{\rho_{c,tp} \gamma_{M0}}{\gamma_{M1}} = 0.774$

Side plates check

x-direction

$\sigma_{max} := -f_y(t_p)$

$\psi(z) := \left[\frac{\sigma_{min}}{\sigma_{max}} + \left(\sigma_{max} \right) \right] - 1$

$\psi_{sp,x} := \frac{\psi(h_w - y_{fg} - t_p)}{\psi(h_w - y_{fg})} = -0.096$

$\sigma_{x,sup} := \frac{M_{Ed}}{I_{yd}} \left(h_w - y_{fg} - y_{fg} \right) = 262.999\,MPa$

$\sigma_{x,bot} := \frac{M_{Ed}}{I_{yd}} \left(h_w - y_{fg} - h_b - y_{fg} \right) = -25.217\,MPa$

$\psi_{sp,x} := \max \left(3 + \psi_{sp,x}, 0 \right) = 2.904$ $\psi_{sp1} := \frac{\sigma_{x,bot}}{\sigma_{x,sup}} = -0.096$

$\rho_{p,sp,x} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\,(3+\psi_{sp,x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

$k_{\sigma,sp,x} := \begin{cases} 1 & \text{if } \psi_{sp,x} \geq 1 \\ \frac{0.2}{\left[1.05 + \psi_{sp,x} \right]} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\,\psi_{sp,x} + 9.78\,\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98\,(1-\psi_{sp,x})^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 95.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 8.503$

$\sigma_{cr,p,sp,x} := \sigma_{E,sp,x} k_{\sigma,sp,x} = 1.065 \times 10^3\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,x} := \frac{\pi^2\,E\,t_s^2}{12\,(1-\nu^2)\,s_b^2} = 4.745\,MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5 \left[1 + \alpha_{sp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 0.719$

$\chi_{sp} := \min \left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1 \right) = 0.892$

Interpolation

$\xi_{sp,x} := \min \left(\max \left(\frac{\sigma_{cr,sp,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0 \right), 1 \right) = 1$

$\rho_{c,sp,x} := \begin{cases} \left[\left(\rho_{p,sp,sp} - \chi_{sp} \right) k_{sp,x} \left(2 - \xi_{sp,x} \right) + \chi_{sp} \right] & \text{if } \frac{\left[\left(\rho_{p,sp,sp} - \chi_{sp} \right) k_{sp,x} \left(2 - \xi_{sp,x} \right) + \chi_{sp} \right] t_{st}}{t_{bridge}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp,sp} - \chi_{sp} \right) k_{sp,x} \left(2 - \xi_{sp,x} \right) + \chi_{sp} \right] t_{st}}{t_{bridge}} > 1 \end{cases} = 1$

z-direction

$\psi_{sp,z} = 1$

$\alpha_{p,sp} = 0.34$ $\lambda_{p0,sp} = 0.7$

$\phi_{p,sp} = 0.5 \left[1 + \alpha_{p,sp} \left(\lambda_p - \lambda_{p0,sp} \right) + \lambda_p^2 \right] = 0.78$

$\rho_{p,sp,z} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{p,sp} + \sqrt{\phi_{p,sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,p,sp,z} = 4$

$\sigma_{E,sp,z} := \frac{\pi^2\,E\,t_s^2}{12\,(1-\nu^2)\,s_b^2} = 4.745\,MPa$

$\sigma_{cr,p,sp,z} := k_{\sigma,p,sp,z} \sigma_{E,sp,z} = 18.98\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,z} := \frac{\pi^2\,E\,t_s^2}{12\,(1-\nu^2)\,s_b^2} = 125.3\,MPa$

Interpolation

$\xi_{sp,z} := \min \left(\max \left(\frac{\sigma_{cr,p,sp,z}}{\sigma_{cr,c,sp,z}} - 1.0 \right), 1 \right) = 0$

$\rho_{c,sp,z} := \begin{cases} \left[\left(\rho_{p,sp,sp} - \chi_{sp} \right) k_{sp,z} \left(2 - \xi_{sp,z} \right) + \chi_{sp} \right] & \text{if } \frac{\left[\left(\rho_{p,sp,sp} - \chi_{sp} \right) k_{sp,z} \left(2 - \xi_{sp,z} \right) + \chi_{sp} \right] t_{st}}{t_{bridge}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp,sp} - \chi_{sp} \right) k_{sp,z} \left(2 - \xi_{sp,z} \right) + \chi_{sp} \right] t_{st}}{t_{bridge}} > 1 \end{cases} = 0.892$

Top plate stiffeners - check

x-direction

$\rho_{p,tp,sp} := \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$\psi_{sp,x} := \frac{\psi(h_w - y_{fg} - h_{st})}{\psi(h_w - y_{fg})} = 0.606$

$$\sigma_{E,cpn,x} := \frac{\pi^2 \cdot E \cdot t_{sx}^2}{12\left(1-\nu^2\right) \cdot b_{sx}^2} = 2.479 \cdot 10^3 \ \text{MPa}$$

$$k_{\sigma,cpn,x} := \begin{cases} 1 & \text{if } \psi_{cpn,x} \geq 1 \\ \frac{8.2}{1.05 + \psi_{cpn,x}} & \text{if } 1 > \psi_{cpn,x} > 0 \\ 7.81 & \text{if } \psi_{cpn,x} = 0 \\ 7.81 - 6.29\psi_{cpn,x} + 9.78\psi_{cpn,x}^2 & \text{if } 0 > \psi_{cpn,x} > -1 \\ 23.9 & \text{if } \psi_{cpn,x} = -1 \\ 5.98\left(1-\psi_{cpn,x}\right)^2 & \text{if } -1 > \psi_{cpn,x} > -3 \\ 95.68 & \text{if } \psi_{cpn,x} < -3 \end{cases} = 4.089$$

$$\sigma_{cr,cpn,x} := k_{\sigma,cpn,x} \cdot \sigma_{E,cpn,x} = 1.014 \cdot 10^4 \ \text{MPa}$$

Column-like behaviour

$$\sigma_{cr,c,cpn,x} := \frac{\pi^2 \cdot E \cdot t_{sx}^2}{12\left(1-\nu^2\right) \cdot h_{sx}^2} = 12.147 \ \text{MPa}$$

$$\chi_{cpn} := \chi_{cp}$$

Interpolation

$$\xi_{cpn,x} := \min\left(\max\left(\frac{\sigma_{cr,cpn,x}}{\sigma_{cr,c,cpn,x}} - 1.0, 0\right), 1\right) = 1$$

$$\sigma_{cr,cpn,x} := \begin{cases} \left[\left(\sigma_{cr,cpn} - \chi_{cpn}\right)\psi_{cpn,x}\left(2 - \psi_{cpn,x}\right) - \right] \cdot \xi_{cpn,x} + \chi_{cpn} & \text{if } \frac{\left[\left(\sigma_{cr,cpn} - \chi_{cpn}\right)\psi_{cpn,x}\left(2 - \psi_{cpn,x}\right) - \right]\sigma_{sdi}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\sigma_{cr,cpn} - \chi_{cpn}\right)\psi_{cpn,x}\left(2 - \psi_{cpn,x}\right) + \chi_{cpn}\right]\sigma_{sdi}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,cp}}{\frac{\sigma_{cr,cpn,x} \cdot f_y[t_s]}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,cp}}{\frac{\tau_{cr,cpn} \cdot f_y[t_s]}{\gamma_{M1}}}\right)^2 = 0.983$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,cp}}{\frac{\sigma_{cr,cpn,x} \cdot f_y[t_w]}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,cp}}{\frac{\sigma_{cr,cpn,x} \cdot f_y[t_w]}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,cp}}{\frac{\sigma_{cr,cpn,x} \cdot f_y[t_w]}{\gamma_{M1}}}\right)\left(\frac{\sigma_{x,Ed,cp}}{\frac{\sigma_{cr,cpn,x} \cdot f_y[t_w]}{\gamma_{M1}}}\right) + 3\left(\frac{\tau_{Ed,cp}}{\frac{\tau_{cr,cpn} \cdot f_y[t_w]}{\gamma_{M1}}}\right)^2 = 0.627$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,cp}}{\frac{\sigma_{cr,cpn,x} \cdot f_y[t_{sx}]}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,cp}}{\frac{\tau_{cr,cpn} \cdot f_y[t_{sx}]}{\gamma_{M1}}}\right)^2 = 0.553$$

EWM - Changed side plate thickness

This chapter covers the EWM when changing the side plate thickness to 14 mm

Geometry

Amount of:

Upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{su} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350\,mm$

Side plate: $h_s := 389.2mm$ $t_s := 14mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su}\cos\left(21.12\frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{tx} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_{sd} := t_d \cdot b_d$ $A_s := 2t_s h_s$ $A_w := 2t_w h_w$ $A_b := t_b \cdot b_b$

$A_{sus} := n_{su} t_{su} h_{su}$ $A_{sus} := n_{su} t_{su} h_{su}$

$A_{tot} := A_{sd} + A_s + A_w + A_b + A_{sus} + A_{ws} = 0.123\,m^2$

$A_c := A_{tot} + A_{sd} = 0.055\,m^2$

Change of center of gravity, z_{sp}

$\Delta I = \sum_{i=1}^{n} I_i \cdot z_i$

(From top to centre of gravity)

$$z_{sp} := \frac{\begin{pmatrix} A_{sd}\frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ + A_b\left(t_d + h_s + h_{wy}\right) + A_{sus}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right] \end{pmatrix}}{A_{tot}} = 361.088\,mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12\frac{\pi}{180}\right)s_{su0} = 7.206\,mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so}\frac{t_{so}h_{so}^3}{12} + n_{so}t_{so}h_{so}\left(t_d + \frac{h_{so}}{2} - z_{sp}\right)^2 = 1.626 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506\,10^{-6}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2n_{su}t_{su}h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \dots = 9.273 \times 10^{-4}\,m^4$$
$$+ 2n_{su}t_{su}h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \dots$$
$$+ 2n_{su}t_{su}h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w h_w\left[h_s - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{sp}\right]^2 = 3.137 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d\left(\frac{t_d}{2} - z_{sp}\right)^2 = 4.438 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b h_b^3}{12} + b_b h_b\left(h_s - z_{sp} - \frac{t_b}{2}\right)^2 = 4.342 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s h_s^3}{12} + 2t_s h_s\left(t_d + \frac{h_s}{2} - z_{sp}\right)^2 = 4.044 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{sp}} z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

$\sigma_{2.neg}(z_1, z_2) := \min\left(\sigma_{neg}(z_1), \sigma_{neg}(z_2)\right)$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{sp}} z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

$\sigma_{2.pos}(z_1, z_2) := \min\left(\sigma_{pos}(z_1), \sigma_{pos}(z_2)\right)$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$$k_{\sigma.4.2j}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4\varepsilon(\psi)\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max(3 + \psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2) Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 399.2\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.109$

$$\text{Condition} := \begin{cases} 62\varepsilon\left(f_y\right)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \le -1 \\ \frac{42\varepsilon\left(f_y\right)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$$

$$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.608$

$s_{c.so} := s_{so} - s_{so}$ [Table 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p\left(k_{\sigma.4.1}(\psi_s), b_w, t_s\right) = 0.41$

$\rho_s := \rho_{4.2}\left(\lambda_{p.s}, \psi_s\right) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{sp} = 0.038\,m$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 351.088\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s\left(h_{s.c}\right) = 351.088\,mm$

$h_{s.eff.c1} := 0.4\,h_{s.eff.c} = 140.435\,mm$

$h_{s.eff.c2} := 0.6\,h_{s.eff.c} = 210.653\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2\,mm$

$A_{s.eff.c} := 2\cdot h_{s.eff.c} t_s = 9.83 \times 10^3\,mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right)t_s = 1.09 \times 10^4\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s)\frac{E\cdot\pi^2\cdot s_s^2}{12\left(1 - \nu^2\right)h_s^2} = 2.114 \times 10^3\,MPa$ EC1993-1-5 eq. A.1

Stiffeners

$h_{st} := t_d + h_{so} = 150\,mm$

$h_{sb} := t_d + h_{su} = 130\,mm$

$\psi_{so} := \frac{\sigma_{neg}(z_s)}{\sigma_{neg}(z_t)} = 0.601$

$k_{\sigma,4.2it}(\psi_{so}) = 0.614$ Higher stress close to support

Class4_so := "Class4" if $\frac{b_{so}}{t_{so}} > 21 \cdot \varepsilon \cdot (k_{\sigma} \sqrt{k_{\sigma,4.2it}(\psi_{so})})$ = "Lower than class 4" otherwise

$\lambda_{p,so} := \lambda_p(k_{\sigma,4.2it}(\psi_{so}), b_{so}, t_{so}) = 0.483$

$\rho_{d,so} := \rho_{4.2}(\lambda_{p,so}, \psi_{so}) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so,eff} := \rho_{d,so} \cdot b_{so} = 140 \cdot mm$

$A_{so,eff} := n_{so} \cdot t_{so} \cdot b_{so,eff} = 2.016 \times 10^{-3} \cdot m^2$

Deck

$\frac{b_{so}}{t_d} = 35$ $42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ = "Class4" if $\frac{b_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4" "Lower than class 4" otherwise

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} \cdot m$ $\psi_d := \psi_{neg}(z_d, z_s) = 1$

$b_{so} := b_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma,d} := k_{\sigma,4.1}(\psi_d) = 4$

$\lambda_{p,d} := \lambda_p(k_{\sigma,d} \cdot k_{\tau,so} \cdot t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p,d}, \psi_d) = 0.962$ Internal cross-sectional parts

$b_{so,eff} := \rho_d \cdot b_{so} = 321.461 \cdot mm$

Areas:

$A_{so} := n_{so} \cdot b_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (b_{so,eff} + t_d) = 5.31 \times 10^{-4} \cdot m^2$

$A_{c,eff,loc} := n_{so} \cdot b_{so,eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (b_{so,eff} + t_d) = 5.053 \times 10^{-4} \cdot m^2$

More than three longitudinal stiffeners --> $k_{\tau,so}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-7} \cdot m^4$

$y_{sp,sl} := \frac{n_{so} \cdot t_{so} \cdot b_{so} \cdot \left(t_d + \frac{b_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot b_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp,sl}\right)^2\right] + \left[\frac{n_{so} \cdot t_{so} \cdot b_{so}^3}{12} + n_{so} \cdot b_{so} \cdot t_{so} \cdot \left(t_d + \frac{b_{so}}{2} - y_{sp,sl}\right)^2\right] = 1.052 \times 10^{-8} \cdot m^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143$ $\alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot b_{so} \cdot t_{so} = 0.02 \cdot m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035 \cdot m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$\beta_{A,c} := \frac{A_{c,eff,loc}}{A_c} = 0.952$ $\cdot 1$ if $\alpha \le \frac{4}{\sqrt{\gamma}} = 639.127$

$\lambda_{sp} := \sqrt{\frac{\beta_{A,c} \cdot f_y(t_d)}{\sigma_{cr,p}}} = 0.584$ herwise

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ orthotropic plate behaviour

$E = 2.1 \times 10^5 \cdot MPa$ $\nu = 0.3$ $\sigma_{cr,p} = 236.836 \cdot MPa$ EC1993-1-5 eq.A.1

$A_{c,eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c,eff,loc} + n_{so,eff} \cdot t_d = 5.375 \times 10^{-4} \cdot m^2$

Bucking strength

$N_{Rd} := A_{c,eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^4 \cdot kN$

Column like behaviour

$y_{sp,sl,c} := \frac{t_{so} \cdot b_{so} \cdot \left(t_d + \frac{b_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot b_{so} + n_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl,c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp,sl,c}\right)^2\right] + \left[\frac{t_{so} \cdot b_{so}^3}{12} + b_{so} \cdot t_{so} \cdot \left(t_d + \frac{b_{so}}{2} - y_{sp,sl,c}\right)^2\right] = 1.137 \times 10^{-7} \cdot m^4$

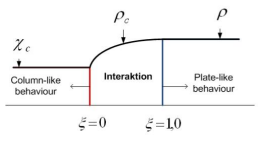
Bucking resistance stiffener

$A_{sl} := t_{so} \cdot b_{so} + n_{so} \cdot t_d = 5.74 \times 10^{-3} \cdot m^2$

$A_{sl,eff} := t_{so} \cdot b_{so,eff} + n_{so,eff} \cdot t_d = 5.455 \times 10^{-3} \cdot m^2$

$\beta_c := \frac{A_{sl,eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr,sl} := \frac{\pi^2 \cdot E \cdot I_{sl,c}}{A_{sl} \cdot a^2} = 1.026 \times 10^3 \cdot MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr,sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$\bar{e} := \max\left[\left(y_{sp,sl} - \frac{t_d}{2}\right), \left(t_d + \frac{b_{so}}{2} - y_{sp,sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{\bar{e}}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr,p}}{\sigma_{cr,sl}} - 1, 1\right), 0\right) = 0$

ξ is zero--> column like behaviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{axst} := n_{so} \cdot t_{so,eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{b_{so,eff}}{2}\right) = -1.57 \times 10^{-6} \cdot kN$

$\sigma_{com,Ed} := \frac{F_{axst}}{n_{so} \cdot t_{so,eff} \cdot t_d + A_{so}} = -319.804 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{so})}{\gamma_M} = 281.032 \cdot MPa$

$K_{A.2.1}$ = "OK" if $\frac{\rho_c \cdot f_y(t_d)}{\gamma_M} > \sigma_{com,Ed}$ = "OK" "EJ OK" otherwise

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d,eff} := n_{so} \cdot t_d \cdot s_{so,eff} = 0.032 \cdot m^2$

$A_{tot,eff} := A_{so,eff} + A_{d,eff} + A_w + A_{so} + A_b + A_{s,eff} = 0.12 \cdot m^2$

$z_{sp,eff,neg} := \frac{\begin{aligned} &A_{d,eff} \cdot \frac{t_d}{2} + A_{so,eff} \cdot \left(t_d + \frac{s_{so,eff}}{2}\right) \cdots \\ &+ 2 \cdot \left[t_s \cdot b_{s,eff,c1} \cdot \left(t_d + \frac{b_{s,eff,c1}}{2}\right) \cdots \right. \\ &\left. + t_s \cdot b_{s,eff,c2} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c2}}{2}\right) + t_s \cdot b_{s,t} \cdot \left(t_d + b_s - \frac{b_{s,t}}{2}\right)\right] \cdots \\ &+ A_w \cdot \left(t_d + b_s + \frac{b_{wy}}{2}\right) + A_{so} \cdot \left[t_d + b_s + \frac{2}{3}\left(b_{wy} - s_{so,b}\right) - \frac{b_{so,y}}{2}\right] \cdots \\ &+ A_b \cdot \left(b_{tb} - \frac{b_b}{2}\right) \end{aligned}}{A_{tot,eff}} = 369.556 \cdot mm$

$I_{y,s,eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot b_{s,eff,c1}^3}{12} + t_s \cdot b_{s,eff,c1} \cdot \left(t_d + \frac{b_{s,eff,c1}}{2} - z_{sp,eff,neg}\right)^2 \cdots \\ &+ \frac{t_s \cdot b_{s,eff,c2}^3}{12} + t_s \cdot b_{s,eff,c2} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c2}}{2} - z_{sp,eff,neg}\right)^2 \cdots \\ &+ \frac{t_s \cdot b_{s,t}^3}{12} + t_s \cdot b_{s,t} \cdot \left(t_d + b_s - \frac{b_{s,t}}{2} - z_{sp,eff,neg}\right)^2 \end{aligned}\right] = 4.341 \times 10^{-4} \cdot m^4$

$I_{y,d,eff} := \frac{n_{s,so} \cdot s_{so,eff} \cdot t_d^3}{12} + n_{s,so} \cdot t_d \cdot s_{so,eff} \cdot \left(\frac{t_d}{2} - z_{sp,eff,neg}\right)^2 = 4.273 \times 10^{-7} \cdot m^4$

$I_{y,eff,neg} := I_{y,st,so} + I_{y,st,su} + I_{y,w} + I_{y,b} + I_{y,s} + I_{y,s,eff} + I_{y,d,eff} = 0.015 \cdot m^4$

Negative moment

Effective cross-section with negative moment(lower flange in tension)

Area

$A_{sl,eff,neg} := A_{d,eff}$

$A_{c,eff,loc} = 5.053 \times 10^{-4} \cdot m^2$

$A_{c,eff,loc,mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com,Ed} \cdot \gamma_M} = -5.044 \times 10^{-3} \cdot m^2$ (A.3)

$A_{sl,eff,neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

effective moment of inertia

$A_{eff,tot} := A_{c,eff} + A_w + A_{so} + A_b = 1.109 \times 10^{-3} \cdot m^2 = 432.173 \cdot in^2$

According to EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with similar area reduction factors.

$t_{d,eff} := \begin{cases} \frac{A_{c,eff,loc,mod}}{A_{c,eff,loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.916 \cdot mm$

$y_{sp,eff,neg} := \frac{\begin{aligned} &n_{so} \cdot b_{so,eff} \cdot t_{so} \cdot \left(t_{d,eff} + \frac{b_{so}}{2}\right) + s_{so,eff} \cdot t_{d,eff} \cdot \frac{t_{d,eff}}{2} \cdots \\ &+ n_{so} \cdot s_{so,eff} \cdot t_{d,eff} \cdot \left(\frac{t_{d,eff}}{2}\right) + t_s \cdot b_{s,eff,c1} \cdot \left(t_d + \frac{b_{s,eff,c1}}{2}\right) \cdots \\ &+ t_s \cdot b_{s,eff,c2} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c2}}{2}\right) + t_s \cdot b_{s,t} \cdot \left(t_d + b_s - \frac{b_{s,t}}{2}\right) \cdots \\ &+ A_w \cdot \left(t_d + b_s + \frac{b_{wy}}{2}\right) + A_{so} \cdot \left[t_d + b_s + \frac{2}{3}\left(b_{wy} - s_{so,b}\right) - \frac{b_{so,y}}{2}\right] \cdots \\ &+ A_b \cdot \left(b_{tb} - \frac{b_b}{2}\right) \end{aligned}}{A_{eff,tot}} = 386.486 \cdot mm$

Effective moment of inertia:

$I_{y,d,eff,fin} := \frac{n_{s,so} \cdot s_{so,eff} \cdot t_d^3}{12} + t_d \cdot n_{s,so} \cdot s_{so,eff} \cdot \left(\frac{t_d}{2} - z_{sp,eff,neg}\right)^2 = 4.679 \times 10^{-7} \cdot m^4$

$\delta_{so,fin} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{so,fin} = 7.206 \cdot mm$

Stiffeners overlunder:

$I_{y,st,su,fin} := n_{so} \cdot \frac{s_{so,eff} \cdot t_{so}^3}{12} + n_{so} \cdot s_{so} \cdot t_{so} \cdot s_{so,eff} \cdot \left(t_d + \frac{s_{so,eff}}{2} - z_{sp,eff,neg}\right)^2 = 1.927 \times 10^{-7} \cdot m^4$

$I_{y,st,su,1,fin} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$I_{y,st,su,fin} := I_{y,st,su,1,fin} \cdots = 8.015 \times 10^{-4} \cdot m^4$
$+ 2 \cdot t_{so} \cdot b_{so} \cdot \left[t_d + b_s + \frac{1}{2}\left(b_{wy} - s_{so,b}\right) - \frac{b_{so,y}}{2} - z_{sp,eff,neg}\right]^2 \cdots$
$+ 2 \cdot t_{so} \cdot b_{so} \cdot \left[t_d + b_s + \frac{2}{3}\left(b_{wy} - s_{so,b}\right) - \frac{b_{so,y}}{2} - z_{sp,eff,neg}\right]^2 \cdots$
$+ 2 \cdot t_{so} \cdot b_{so} \cdot \left[t_d + b_s + \left(b_{wy} - s_{so,b}\right) - \frac{b_{so,y}}{2} - z_{sp,eff,neg}\right]^2$

Web:

$I_{y,w,1,fin} := 2 \cdot 0.00031 \cdot m^4$ from FEM

$I_{y,w,fin} := I_{y,w,1,fin} + 2 \cdot t_w \cdot b_w \cdot \left[b_{tb} - \frac{b_b}{2} - \frac{1}{2}\left(b_{wy}\right) - z_{sp,eff,neg}\right]^2 = 2.701 \times 10^{-7} \cdot m^4$

Bottom flange:

$I_{y,b,fin} := \frac{b_b \cdot b_b^3}{12} + t_b \cdot b_b \cdot \left(b_{tb} - z_{sp,eff,neg} - \frac{b_b}{2}\right)^2 = 3.929 \times 10^{-7} \cdot m^4$

Side plate:

$I_{y,s,fin} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot b_{s,eff,c1}^3}{12} + t_s \cdot b_{s,eff,c1} \cdot \left(t_d + \frac{b_{s,eff,c1}}{2} - z_{sp,eff,neg}\right)^2 \cdots \\ &+ \frac{t_s \cdot b_{s,eff,c2}^3}{12} + t_s \cdot b_{s,eff,c2} \cdot \left(t_d + b_{s,c} - \frac{b_{s,eff,c2}}{2} - z_{sp,eff,neg}\right)^2 \cdots \\ &+ \frac{t_s \cdot b_{s,t}^3}{12} + t_s \cdot b_{s,t} \cdot \left(t_d + b_s - \frac{b_{s,t}}{2} - z_{sp,eff,neg}\right)^2 \end{aligned}\right] = 4.981 \times 10^{-4} \cdot m^4$

$I_{y,eff,neg,fin} := I_{y,d,eff,fin} + I_{y,st,so,fin} + I_{y,st,su,fin} + I_{y,w,fin} + I_{y,b,fin} + I_{y,s,fin} = 1.453 \times 10^{-3} \cdot m^4$

Load

Load capacity:

$q_{_}M := 94 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_{_}M := q_{_}M \cdot b_d = 329 \frac{kN}{m}$

$M_{Ed} := \frac{Q_{_}M \cdot l_{bridge}^2}{8} = 1.332 \times 10^4 \cdot kN \cdot m$

$q_{.}V := 157\frac{kN}{m^2}$ The load capacity is q_v when checking the shear capacity.

$Q_{.}V := q_{.}V \cdot b_{rd} = 549.5\frac{kN}{m}$

$V_{Ed} := \frac{Q_{.}V \cdot b_{bridge}}{2} = 4.946 \times 10^3 kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{sp.eff.neg}} = 0.038\,m^3$

$M_{Rd} := W_{y.eff}\frac{f_y(t_d)}{\gamma_{M1}} = 1.335 \times 10^4 kN \cdot m$

Moment capacity

$\frac{M_{Ed}}{M_{Rd}} = 0.998$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot t_{sw} = 0.293m$

$\alpha_w := \min\left(\frac{h_{ww}}{h_w}, \frac{180}{\pi}\right) = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{tw}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{tw}}{2} = 0.503m$

$a_{st.z.2} := a_{st.z.1} + s_{tw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913m$

$a_{st.z.3} := a_{st.z.2} + s_{tw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 \cdot t_{sw} \cdot h_{tw}\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3} m^4$

Distance from z-axis to middle of the web area which is 15ct each side of the stiffeners:

$a_{web.z} := \frac{h_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{tw}}{2} = 0.525m$

$a_{st.z.1} := a_{st.z.1} + s_{tw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935m$

$a_{st.z.2} := a_{st.z.2} + s_{tw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346m$

Moment of inirtia of 15st of the web z-direction

$I_{sw.z} := 2b_{web} \cdot s_w\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$

$I_{st.z.tot} := I_{st.z} + I_{sw.z} = 0.029m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00\left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34\left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.491$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{sl} = 0$

$\lambda_{w1} := \frac{37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_{\tau 1}}}{t_w} = 0.93$ SS-EN 1993-1-5 5.3 (1) och (2) ger: EC1-3-5 eq.5.6

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18\frac{I_{st.z.tot}}{t_w^3 \cdot h_{web}}}{\alpha^2} + 2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_w, t_{sw}\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^3$ 1)

$k_{\tau sl} := \max\left(9\left(\frac{\alpha_w}{\alpha}\right)\sqrt{\frac{I_{st.z.tot}}{\min\left(t_w, t_{sw}\right)^3 \cdot h_{web}}}, \left(\frac{2.1}{\min\left(t_w, t_{sw}\right)}\right)\sqrt{\frac{I_{st.z.tot}}{h_w}}\right) = 4.782 \times 10^3$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\gamma_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_w, t_{sw}\right)} \leq 31 \cdot \frac{\varepsilon(t_w)}{\eta}\sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{\left(37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}\right)}, \lambda_{w1}\right) = 0.39$ Enligt SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ SS-EN 1993-1-5 5.3 Table 5.1, stiff and stiffener.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M1} := 1.00$ s.SB3 bärande konstruktioner

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.479 \times 10^3 kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.946 \times 10^3 kN$

$\frac{V_{Ed}}{2V_{bw.Rd}} = 0.997$

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4, SB3 "bärande konstruktioner"

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{tf.eff.neg} := z_{sp.eff.neg} - \frac{t_f}{2} = 381.486 \cdot mm$

$z_{sf.eff.neg} := h_w - \frac{h_b}{2} - z_{sp.eff.neg} = 495.514 \cdot mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \cdot z_{tf.eff.neg} \cdot f_y(t_f)}{\gamma_{M0}} \ldots = 7.168 \times 10^3 kN \cdot m$
$+ \frac{A_{sf.eff.neg} \cdot z_{sf.eff.neg} \cdot f_y(t_f)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tf} := \min\left[0.25 + \frac{1.6\,b_f \cdot t_f^2 \cdot f_y(t_f)}{t_f\left(h_b + h_{wp}\right)^2 \cdot f_y(t_f)}\right] = 0.605m$

$c_{sf} := \min\left[0.25 + \frac{1.6\,b_f \cdot h_b^2 \cdot f_y(t_f)}{t_f\left(h_b + h_{wp}\right)^2 \cdot f_y(t_f)}\right] = 0.577m$

$V_{bf.Rd.creed} := \frac{b_f \cdot t_f^2 \cdot f_y(t_f)}{c_{tf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot h_b^2 \cdot f_y(t_b)}{c_{sf} \cdot \gamma_{M1} \cdot \gamma_n} = 302.348 \cdot kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.creed}\left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\,kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.web} := V_{bw.Rd} + V_{bf.Rd} = 2.479 \times 10^3 kN$

Check:

$\frac{V_{Ed}}{2 \cdot V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 \cdot V_{b.Rd}} = 0.997$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.sf.eff.fin} + I_{y.b.fin} = 8.607 \times 10^{-3} m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{wo.eff} \cdot t_d + n_w \cdot s_{wo.eff} \cdot t_{d.eff} + n_w \cdot t_{wo} \cdot t_{d.eff} + A_b = 432.579 \cdot cm^2$

z_{sp} for the flanges effective areas:

$z_{sp.f.eff.neg} := \frac{s_{wo.eff} \cdot t_d \cdot \frac{t_d}{2} + n_w \cdot s_{wo.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \ldots}{+ n_w \cdot t_{wo} \cdot t_{d.eff}\left(\frac{t_{d.eff}}{2}\right) + A_b\left(h_b - \frac{t_b}{2}\right)} = 328.801 \cdot mm$
$A_{eff.f.tot}$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{sp.f.eff.neg}} = 0.026\,m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff}\frac{f_y(t_d)}{\gamma_M} = 9.293 \times 10^3 \cdot kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.335 \times 10^4 kN \cdot m$

Moment capacity check when $q_{.}M = 94\frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.998$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_{.}V = 157\frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2V_{bw.Rd}} = 0.997$

The Reduced stress method

-Changed side plate thickness to 14 mm

Material

$f_{y.0} := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ 335\text{MPa} & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} = 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\text{GPa}$

Geometry

Bottom plate:

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b t_b = 0.016\text{m}^2$

Top plate

$t_{tp} := 10\text{mm}$ $b_{tp} := 3500\text{mm}$ $A_{tf} := b_{tp} t_{tp} = 0.035\text{m}^2$

Side plates:

$t_s := 14\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s t_s = 5.449 \times 10^{-3}\text{m}^2$

Webs:

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w t_w = 0.016\text{m}^2$

$h_{w.k} := 1258\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{st} := 16\text{mm}$ $b_{st} := 140\text{mm}$ $A_{st} := b_{st} t_{st} = 2.24 \times 10^{-3}\text{m}^2$

$s_{st} := 350\text{mm}$ $n_{st} := 9$

Web stiffeners:

$t_{wst} := 12\text{mm}$ $b_{wst} := 120\text{mm}$ $A_{wst} := b_{wst} t_{wst} = 1.44 \times 10^{-3}\text{m}^2$

$s_{wst} := 440\text{mm}$ $n_{wst} := 3$

Whole bridge

$t_{totdge} := 18\text{in}$ $h_t := \frac{h_w}{2} + h_{w.y} + t_s + t_{tp} = 0.89\text{m}$ $t_{sp} := 2\text{m}$

$I_{tot} := 0.01486\text{m}^4$ From grasshopper

$y_{tp} := 0.529006\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_{Ed} := 78\frac{\text{kN}}{\text{m}^2}$

$Q := q_{Ed} \cdot s = 273\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q \cdot L_{bridge}^2}{8} = 1.106 \times 10^4\,\text{kN m}$

$V_{Ed} := \frac{Q \cdot L_{bridge}}{2} = 2.457 \times 10^3\,\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}} \cdot \left(h_w - \frac{t_f}{2} - y_{tp}\right) = 264.875\,\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{b_f t_f}{n_{st}+2} \cdot \left(h_w - \frac{t_f}{2} - y_{tp}\right) = 1.133 \times 10^{-3}\,\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_{tot} \cdot \min(t_f; t_{st})} = 13.378\,\text{MPa}$

Side plates:

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}} \cdot (h_w - t_f - y_{tp}) = 261.155\,\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{n_s t_s} = 9.75\,\text{MPa}$

$S_{sp} := (h_w - t_f - y_{tp}) t_s \left(\frac{h_w - t_f - y_{tp}}{2}\right) + \frac{t_f b_f}{2}\left(h_w - \frac{t_f}{2} - y_{tp}\right) = 9.925 \times 10^{-3}\,\text{m}^3$

$+ \frac{h_{st}}{2} t_{st} \left(h_w - t_f - \frac{h_{st}}{2} - y_{tp}\right)$

$S_{tps} := \frac{\left(h_w - \frac{t_{st} h_{st}}{2} + 2\frac{t_f h_f}{2}\right)\left(h_w - t_f - \frac{h_{st}}{4} - y_{tp}\right)}{n_{st}+2} = 1.479 \times 10^{-3}\,\text{m}^3$

$+ \frac{t_f b_f}{n_{st}+2}\left(h_w - \frac{t_f}{2} - y_{tp}\right)$

$\tau_{Ed.sp} := \frac{V_{Ed} \cdot S_{tps}}{I_{tot} \cdot t_s} = 15.28\,\text{MPa}$

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}} \cdot (h_w - t_f - y_{tp}) = 261.155\,\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.1.Ed} := 4.37\frac{\text{MPa}}{\text{m}^2}$ From a linear stress analysis in Sofistik

$\sigma_{cr} := \sigma_{V.1.Ed} \cdot q = 340.86\,\text{MPa}$

$\alpha_{ult} := \frac{f_y[0]}{\sigma_{cr}} = 1.041$

$\alpha_{cr.1.Ed} := 220.827\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.1.Ed}}{q_{Ed}} = 2.831$

$\lambda_p := \sqrt{\frac{f_{y.0}}{\sigma_{cr}}} = 0.607$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{p.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2 E t_f^2}{12\cdot(1-\nu^2)\cdot b_{tp}^2} = 1.549\,\text{MPa}$

$I_{y.tp} := \frac{b_{tp} t_f^3}{12\cdot(1-\nu^2)} = 3.205 \times 10^{-7}\,\text{m}^4$

$y_{tp.stiff} := \frac{A_{tf}\frac{t_f}{2} + n_{st} A_{st}\left(t_f + \frac{h_{st}}{2}\right)}{A_{tf} + n_{st} A_{st}} = 32.411\,\text{mm}$

$I_{tp.tp} := \frac{b_f t_f^3}{12} + A_{tf}\left(y_{tp.stiff} - \frac{t_f}{2}\right)^2 + \dots = 1.052 \times 10^{-8}\,\text{mm}^4$

$+ n_{st}\left[\frac{t_{st} h_{st}^3}{12} + A_{st}\left(y_{tp.stiff} - t_f - \frac{h_{st}}{2}\right)^2\right]$

$y_{tp} := \frac{I_{tp.tp}}{y_{tp.tp}} = 328.143$

$\delta_{tp} := \frac{n_{st} A_{st}}{A_{tf}} = 0.576$

$\alpha_{tp} := \min\left(\frac{2\pi}{b_{tp}}; 0.5\right) = 0.571$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt{\gamma_{tp}} \\ \frac{4\cdot(1+\sqrt{\gamma_{tp}})}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt{\gamma_{tp}} \end{cases} = 639.127$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp} \cdot \sigma_{E.tp} = 990.256\,\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1.outf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}} b_{tp} = 0.175\text{m}$

$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}} b_{tp} = 0.175\text{m}$ Parts of the top plate that contributes to the stiffener

$b_{tf.1.tp} := b_{1.outf.tp} + b_{2.sup.tp} + t_{wst} = 0.35\text{m}$

$y_{tp.tf.1.tp} := \frac{t_f b_{tf.1.tp}\frac{t_f}{2} + A_{wst}\left(t_f + \frac{h_{wst}}{2}\right)}{t_f b_{tf.1.tp} + A_{wst}} = 0.034\text{m}$ combined center of gravity for stiffener and contributing top plate.

$I_{tf.1.tp} := \frac{b_{tf.1.tp} t_f^3}{12} + b_{tf.1.tp} t_f\left(y_{tp.tf.1.tp} - \frac{t_f}{2}\right)^2 + \frac{t_{wst} h_{wst}^3}{12} + A_{wst}\left(y_t + \frac{h_{wst}}{2} - y_{tp.tf.1.tp}\right)^2$

Combined moment of inertia

$\sigma_{cr.tf.tp} := \frac{\pi^2 E I_{tf.1.tp}}{A_{tf.1.tp} \cdot s_{st}^2} = 1.026 \times 10^3\,\text{MPa}$

Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.s} = \sigma_{cr.tf}$

$\sigma_{cr.c.tp} := \sigma_{cr.tf.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{tf.1.tp}}{A_{tf.1.tp}}} = 0.045\text{m}$

$e_{tp} := \max\left(y_{tp.tf.1.tp} - t_f - t_f + \frac{h_{wst}}{2} - y_{tp.tf.1.tp}\right) = 0.046\text{m}$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$ Stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e.tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.802$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\left(\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}\right)}; 1\right) = 0.753$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.tp.tp}}{\sigma_{cr.c.tp}} - 1.0\right); 1\right) = 0$

$\rho_{c.tp} := \left| \begin{array}{l} \left[\left(\rho_{p.tp} - \chi_{tp}\right)\xi_{tp}\left(2 - \xi_{tp}\right) + \chi_{tp}\right] \cdot \frac{\left[\left(\rho_{p.tp} - \chi_{tp}\right)\xi_{tp}\left(2 - \xi_{tp}\right) + \chi_{tp}\right] n_{st} A_{st}}{\gamma_{M1}} < 1 \\ 1 \quad \text{if} \quad \frac{\left[\left(\rho_{p.tp} - \chi_{tp}\right)\xi_{tp}\left(2 - \xi_{tp}\right) + \chi_{tp}\right] n_{st} A_{st}}{\gamma_{M1}} > 1 \end{array} \right. = 0.753$

Shear

$\eta_t := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta_t & \text{if } \lambda_p < \frac{0.83}{\eta_t} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_t} \le \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$

$\frac{\rho_{c.tp} n_{st} A_{st}}{\gamma_{M1}} = 0.785$

Side plates check

x-direction

$\sigma_{max} := f_y[\psi_{tp}]$

$\sigma(z) := \left[\frac{-\sigma_{max}}{(\psi_{tp}+1)} \cdot z + \sigma_{max}\right] - 1$

$\psi_{sp.x} := \frac{\sigma\left(h_w - t_f - h_f\right)}{\sigma\left(h_w - t_f\right)} = -0.109$

$\sigma_{x.sup} := \frac{M_{Ed}}{I_{tot}}\left(h_w - t_f - y_{tp}\right) = 261.155\,\text{MPa}$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\left(h_w - t_f - h_f - y_{tp}\right) = -28.427\,\text{MPa}$

$\psi_{sp.k.x} := \max\left(3 \cdot \psi_{sp.x}; 0\right) = 2.891$ $\psi_{sp.x} := \frac{\sigma_{x.bot}}{\sigma_{x.sup}} = -0.109$

$k_{\sigma.sp.x} := \begin{cases} 8.2 & \text{if } \psi_{sp.x} \ge 1 \\ \frac{8.2}{(1.05+\psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29\psi_{sp.x} + 9.78\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot(1-\psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} > -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.611$

$\sigma_{cr.sp.x} := \sigma_{E.sp.x} k_{\sigma.sp.x} = 2.115 \times 10^3\,\text{MPa}$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot b_w^2} = 9.3\,\text{MPa}$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.727$

$\chi_{sp} := \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)}; 1\right) = 0.888$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.sp.sp.x}}{\sigma_{cr.c.sp.x}} - 1.0\right); 1\right) = 1$

$\rho_{c.sp.x} := \left| \begin{array}{l} \left[\left(\rho_{p.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2 - \xi_{sp.x}\right) - \right] \cdot \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2 - \xi_{sp.x}\right) - \right] n_{sth}}{\gamma_{M1}} < 1 \\ 1 \quad \text{if} \quad \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2 - \xi_{sp.x}\right) + \chi_{sp}\right] n_{sth}}{\gamma_{M1}} > 1 \end{array} \right. = 1$

z-direction

$\psi_{sp.z} := 1$

$\alpha_{sp.sp} := 0.34$ $\lambda_{sp.sp} := 0.7$

$\phi_{sp.sp} := 0.5\left[1 + \alpha_{sp.sp}\left(\lambda_p - \lambda_{sp.sp}\right) + \lambda_p^2\right] = 0.787$

$\rho_{p.z.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\left(\phi_{sp.sp} + \sqrt{\phi_{sp.sp}^2 - \lambda_p^2}\right)} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot b_w^2} = 9.3\,\text{MPa}$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \sigma_{E.sp.z} = 37.205\,\text{MPa}$

Column-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 E i_z^2}{12\cdot(1-\nu^2)\cdot b_w^2} = 245.588\,\text{MPa}$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1.0\right); 1\right) = 0$

$\rho_{c.sp.z} := \left| \begin{array}{l} \left[\left(\rho_{p.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) - \right] \cdot \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) - \right] n_{sth}}{\gamma_{M1}} < 1 \\ 1 \quad \text{if} \quad \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] n_{sth}}{\gamma_{M1}} > 1 \end{array} \right. = 0.888$

Top plate stiffeners - check

x-direction

$\rho_{p.x.tps} := \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$\psi_{sps,x} := \frac{\min(\sigma_{2_x} - \sigma_{1_x}, \sigma_{3_x})}{\max(\sigma_{2_x} - \sigma_{1_x}, \sigma_{3_x})} = 0.601$

$\sigma_{E,sps,x} := \frac{\pi^2 \cdot E \cdot t_{sps}^2}{12 \cdot (1 - \nu^2) \cdot b_{sps}^2} = 2.479 \cdot 10^3 \cdot MPa$

$k_{\sigma,sps,x} := \begin{cases} 1 & \text{if } \psi_{sps,x} \geq 1 \\ \frac{8.2}{1.05 + \psi_{sps,x}} & \text{if } 1 > \psi_{sps,x} > 0 \\ 7.81 & \text{if } \psi_{sps,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sps,x} + 9.78 \cdot \psi_{sps,x}^2 & \text{if } 0 > \psi_{sps,x} > -1 \\ 23.9 & \text{if } \psi_{sps,x} = -1 \\ 5.98 \cdot (1 - \psi_{sps,x})^2 & \text{if } -1 > \psi_{sps,x} > -3 \\ 95.68 & \text{if } \psi_{sps,x} < -3 \end{cases} = 4.145$

$\sigma_{cr,sps,x} := k_{\sigma,sps,x} \cdot \sigma_{E,sps,x} = 1.027 \cdot 10^4 \cdot MPa$

Column-like behaviour

$\sigma_{cr,c,sps,x} := \frac{\pi^2 \cdot E \cdot i_{sps}^2}{12 \cdot (1 - \nu^2) \cdot s_x^2} = 12.147 \cdot MPa$

$\lambda_{sps} := \lambda_{sp}$

Interpolation

$\xi_{sps,x} := \min\left(\max\left(\frac{\sigma_{cr,sps,x}}{\sigma_{cr,c,sps,x}} - 1.0\right), 1\right) = 1$

$\rho_{c,sps,x} := \begin{cases} \left[\left(\rho_{sps,ips} - \chi_{sps}\right) \cdot k_{sps,x} \cdot \left(2 - \xi_{sps,x}\right) - 1\right] \cdot \frac{\lambda_{sps}}{\chi_{sps}} & \text{if } \frac{\left[\left(\rho_{sps,ips} - \chi_{sps}\right) \cdot k_{sps,x} \cdot \left(2 - \xi_{sps,x}\right) - 1\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{sps,ips} - \chi_{sps}\right) \cdot k_{sps,x} \cdot \left(2 - \xi_{sps,x}\right) + \chi_{sps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Top plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.984$

Side plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right) = 0.576$
$+ 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2$

Top plate stiffeners check

$\left(\frac{\sigma_{x,Ed,sps}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sps}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.545$

C.5 Changed side plate height

CCLXVII 470 mm side plate EWM calculation.
CCLXX 470 mm side plate RSM calculation.
CCLXXII 550 mm side plate EWM calculation.
CCLXXV 550 mm side plate RSM calculation.
CCLXXVII 630 mm side plate RSM calculation.
CCLXXX 630 mm side plate EWM calculation.
CCLXXXII 710 mm side plate RSM calculation.
CCLXXXV 710 mm side plate EWM calculation.
CCLXXXVII 790 mm side plate RSM calculation.
CCXC 790 mm side plate EWM calculation.
CCXCII 870 mm side plate RSM calculation.
CCXCV 870 mm side plate EWM calculation.
CCXCVII 950 mm side plate RSM calculation.
CCC 950 mm side plate EWM calculation.
CCCII 1030 mm side plate RSM calculation.
CCCV 1030 mm side plate EWM calculation.

EWM - Changed side plate height

This chapter covers the EWM when changing the side plate height to 470 mm.

Geometry

Amount of:

upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s.su} := n_{su} + 1 = 10$

Deck: $h_d := 3500mm$ $t_d := 10mm$

$h_{d.0.5} := 1750mm$

$s_{su} := \frac{h_d}{n_{s.su}} = 350\,mm$

Side plate: $h_s := 470mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $h_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{su} := 140mm$ $t_{su} := 16mm$ $a_{su} := 6mm$

Stiffeners under: $h_{sl} := 120mm$ $t_{sl} := 12mm$ $a_{sl} := 5mm$

$h_{sl.y} := h_{sl}\cos\left(21.12\frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 970.8\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 944.8\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Material parameters:

Material

$A_d := t_d \cdot h_d$ $A_s := 2t_s h_s$ $A_w := 2t_w h_w$ $A_b := t_b h_b$

$A_{su} := n_{su} t_{su} h_{su}$ if $\ldots$ $A_{sl} := n_{sl} t_{sl} h_{sl}$ C1993-1-1 tab. 3.1

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.123\,m^2$

$A_c := A_{su} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d\frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) - \ldots + A_b\left(t_d + h_s + h_{wy}\right) + A_{sl}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sul0}\right) - \frac{h_{sl.y}}{2}\right]}{A_{tot}} = 401.54\,mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{sul0y} := \sin\left(21.12\frac{\pi}{180}\right)s_{sul0} = 7.206\,mm$

Stiffeners over/under:

$I_{y.st.su} := n_{su}\frac{t_{su}h_{su}^3}{12} + n_{su}t_{su}h_{su}\left(t_d + \frac{h_{su}}{2} - z_{tp}\right)^2 = 2.117\times10^{-3}\,m^4$

$I_{y.st.su.1} := 6.1.506\cdot10^{-6}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su}h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{sul0y}\right) - \frac{h_{sl.y}}{2} - z_{tp}\right]^2 - \ldots + 2t_{su}h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sul0y}\right) - \frac{h_{sl.y}}{2} - z_{tp}\right]^2 - \ldots + 2t_{su}h_{su}\left[t_d + h_s + \left(h_{wy} - s_{sul0y}\right) - \frac{h_{sl.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2\cdot0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2h_w t_w\left[h_h - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 3.914\times10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d\left(\frac{t_d}{2} - z_{tp}\right)^2 = 5.504\times10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b t_b^3}{12} + b_b b_b\left(h_w - z_{tp} - \frac{h_b}{2}\right)^2 = 5.041\times10^{-7}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s h_s^3}{12} + 2t_s h_s\left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 4.841\times10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.018\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$ Highest possible stress(in bottom flange) is the yield stress.

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{-\sigma_{max}}{z_{tp}}z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}}z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1-\psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$$k_{\sigma.4.2u}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$$k_{\sigma.4.2l}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(b_p, t, \varepsilon) := \frac{\frac{b_p}{t}}{28.4\cdot\varepsilon\cdot\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max(3+\psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2) Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 480\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.2$

$$Condition := \begin{cases} 62\cdot\varepsilon\left(\psi_s\right)\left(1-\psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42\cdot\varepsilon\left(\psi_s\right)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} \quad = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}\left(\psi_s\right) = 9.463$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p\left(k_{\sigma.4.1}\left(\psi_s\right), h_s, t_s\right) = 0.551$

$\rho_s := \rho_{4.2}\left(\lambda_{p.s}, \psi_s\right) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 78.46\,mm$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 391.54\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s\left(h_{s.c}\right) = 391.54\,mm$

$h_{s.eff.c1} := 0.4\,h_{s.eff.c} = 156.616\,mm$

$h_{s.eff.c2} := 0.6\,h_{s.eff.c} = 234.924\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 470\,mm$

$A_{s.eff.c} := 2\,h_{s.eff.c}\,t_s = 9.397\times10^{3}\,mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right)t_s = 1.128\times10^{4}\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr,p} := k_{\sigma.4.1}\left(\psi_k\right)\frac{E\cdot\pi^2\cdot t_d^2}{12\left(1-\nu^2\right)\cdot b_w^2} = 1.171\times10^3\,MPa$ EC1993-1-5 eqA.1

Stiffeners

$t_{sd} := t_d = 10\,mm$

$b_{sw} := t_d + h_{sw} = 150\,mm$

$\psi_{sw} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.642$

$k_{\sigma.4.2.6}\left(\psi_{sw}\right) = 0.588$ Higher stress close to support

Class4_ := "Class4" if $\frac{h_{sw}}{t_{sw}} > 21\cdot\varepsilon\left(t_{sw}\right)\sqrt{k_{\sigma.4.2.6}\left(\psi_{sw}\right)}$ = "Lower than class 4"; "Lower than class 4" otherwise

$\lambda_{p.sw} := \lambda_p\left(k_{\sigma.4.2.6}\left(\psi_{sw}\right), b_{sw}, t_{sw}\right) = 0.494$

$\rho_{d.sw} := \rho_{4.3}\left(\lambda_{p.sw}, \psi_k\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{sw.eff} := \rho_{d.sw}\cdot b_{sw} = 140\,mm$

$A_{sw.eff} := n_{sw}\cdot t_{sw}\cdot b_{sw.eff} = 2.016\times10^4\,mm^2$

Deck

$\frac{b_{sw}}{t_d} = 35 \quad 42\sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ := "Class4" if $\frac{b_{sw}}{t_d} > 42\sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4"; "Lower than class 4" otherwise

$z_d := \frac{-t_d}{2} = -5\times10^{-3}\,m$ $\psi_{sd} := \psi_{neg}\left(z_d, z_d\right) = 1$

$b_{sw} := b_{sw} - t_{sw}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}\left(\psi_{sd}\right) = 4$

$\lambda_{p.sd} := \lambda_p\left(k_{\sigma.d}, b_{sw}, t_{sd}\right) = 0.723$

$\rho_{sd} := \rho_{4.2}\left(\lambda_{p.sd}, \psi_{sd}\right) = 0.962$ Internal cross-sectional parts

$t_{sw.eff} := \rho_{sd}\cdot t_{sw} = 321.461\,mm$

Areas:

$A_{sw} := n_{sw}\cdot t_{sw}\cdot t_d + n_{sw}\cdot t_{sw}\left(b_{sw.eff} + t_d\right) = 5.31\times10^4\,mm^2$

$A_{c.eff.loc} := n_{sw}\cdot t_{sw.eff}\cdot t_d + n_{sw}\cdot t_{sw}\left(b_{sw.eff} + t_d\right) = 5.053\times10^4\,mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d\cdot t_d^3}{12\left(1-\nu^2\right)} = 3.205\times10^5\,mm^4$

$y_{p.sl} := \frac{n_{sw}\cdot t_{sw}\cdot b_{sw}\left(t_d + \frac{b_{sw}}{2}\right) + b_d\cdot t_d\cdot\frac{t_d}{2}}{n_{sw}\cdot t_{sw}\cdot b_{sw} + b_d\cdot t_d} = 32.411\,mm$

$I_{sl} := \left[\frac{b_d\cdot t_d^3}{12} + b_d\cdot t_d\left(\frac{t_d}{2} - y_{p.sl}\right)^2\right] \ldots + \left[n_{sw}\cdot t_{sw}\cdot b_{sw}^3 + n_{sw}\cdot b_{sw}\cdot t_{sw}\left(t_d + \frac{b_{sw}}{2} - y_{p.sl}\right)^2\right] = 1.052\times10^8\,mm^4$

$\beta_A := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2\left[\left(1+\alpha^2\right)^2\cdot\gamma - 1\right]}{\alpha^2\cdot(\psi+1)\cdot(1+\delta)} & \text{if } \alpha \le \sqrt{\gamma} \\ \frac{a\left(1+\sqrt{\gamma}\right)}{(\psi+1)\cdot(1+\delta)} & \text{otherwise} \end{cases} = 639.127$ 2)

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p}\frac{E\cdot\pi^2\cdot t_d^2}{12\left(1-\nu^2\right)\cdot b_d^2} = 990.256\,MPa$ EC1993-1-5 eqA.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{sw} := \sqrt{\frac{\beta_{A.c}\cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}\left(\lambda_{sp}, \psi\right) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_{sp}, \psi\right)\cdot A_{c.eff.loc} + n_{sw.eff}\cdot t_d = 5.375\times10^4\,mm^2$

Bucking strength

$N_{Rd} := A_{c.eff}\frac{f_y(t_d)}{\gamma_M} = 1.908\times10^4\,kN$

Column like behaviour

$y_{sp.sl.c} := \frac{n_{sw}\cdot t_{sw}\left(t_d + \frac{h_{sw}}{2}\right) + n_{sw}\cdot t_d\cdot\frac{t_d}{2}}{n_{sw}\cdot h_{sw} + n_{sw}\cdot t_d} = 34.268\,mm$

$I_{sl.c} := \left[\frac{n_{sw}\cdot t_d^3}{12} + n_{sw}\cdot t_d\left(\frac{t_d}{2} - y_{p.sl.c}\right)^2\right] \ldots + \left[\frac{t_{sw}\cdot h_{sw}^3}{12} + h_{sw}\cdot t_{sw}\left(t_d + \frac{h_{sw}}{2} - y_{p.sl.c}\right)^2\right] = 1.137\times10^7\,mm^4$

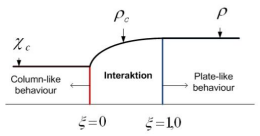
Bucking resistance stiffener

$A_{sw} := t_{sw}\cdot h_{sw} + n_{sw}\cdot t_d = 5.74\times10^3\,mm^2$

$A_{sl.eff} := t_{sw}\cdot h_{sw.eff} + n_{sw.eff}\cdot t_d = 5.455\times10^3\,mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2\cdot E\cdot I_{sl.c}}{A_{sl}\cdot z^2} = 1.026\times10^3\,MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c\cdot f_y(t_{sd})}{\sigma_{cr.sl}}} = 0.573$

calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\,mm$

$e_1 := \max\left(\left[\left(y_{p.sl} - \frac{t_d}{2}\right)\left(t_d + \frac{h_{sw}}{2} - y_{p.sl}\right)\right]\right) = 47.589\,mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e_1}} = 0.522$

$\phi := 0.5\left[1 + \alpha_e\left(\lambda_c - 0.2\right) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaktion mellan de två olika beteendena(column/plate bucking):

$\rho_c := (\rho - \chi_c)\cdot\xi\cdot(2 - \xi) + \chi_c = 0.792$

Checks

Mean stress nxt

The "nxt" in this case consist of two parts in the stiffener and one part in the web.

$F_{avnt} := n_{sw}\cdot t_{sw.eff}\cdot t_d\cdot\sigma_{neg}\left(\frac{t_d}{2}\right) + A_{sw}\cdot\sigma_{neg}\left(t_d + \frac{h_{sw.eff}}{2}\right) = -1.587\times10^4\,kN$

$\sigma_{com.Ed} := \frac{F_{avnt}}{n_{sw}\cdot t_{sw.eff}\cdot t_d + A_{sw}} = -323.35\,MPa$

Check according to A.2.1 (4)

$\frac{\rho_c\cdot f_y(t_d)}{\gamma_M} = 281.032\,MPa$

$K_{A.2.1} := \text{"OK" if } \frac{\rho_c\cdot f_y(t_d)}{\gamma_M} > \sigma_{com.Ed} \text{ = "OK"; "EJ OK" otherwise}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{sw}\cdot t_d\cdot t_{sw.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{sw.eff} + A_{d.eff} + A_w + A_{su} + A_{sb} + A_{s.eff} = 0.12\,m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff}\cdot\frac{t_d}{2} + A_{sw.eff}\left(t_d + \frac{h_{sw.eff}}{2}\right) \ldots \\ &+ 2\left[t_s\cdot h_{s.eff.c1}\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \ldots \right. \\ &\left. + t_s\cdot h_{s.eff.c2}\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s1}\left(t_d + h_s - \frac{h_{s1}}{2}\right)\right] \ldots \\ &+ A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sdy}\right) - \frac{h_{sw.y}}{2}\right] \ldots \\ &+ A_b\left(h_w - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 410.94\,mm$

$I_{y.s.eff} := 2\left[\frac{t_s\cdot h_{s.eff.c1}^3}{12} + t_s\cdot h_{s.eff.c1}\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \ldots + \left[\frac{t_s\cdot h_{s.eff.c2}^3}{12} \ldots + t_s\cdot h_{s.eff.c2}\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \ldots + \left[\frac{t_s\cdot h_{s1}^3}{12} + t_s\cdot h_{s1}\left(t_d + h_s - \frac{h_{s1}}{2} - z_{tp.eff.neg}\right)^2\right]\right]\right] = 5.183\times10^{-4}\,m^4$

Negative moment

Effective cross-section with high tension flange (for shear lag, torsion and distortion)

Area

$I_{y.d.eff.neg} := \frac{b_d\cdot t_{sd.eff}^3}{12} + A_{sd.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.298\times10^{-3}\,m^4$

$I_{y.ext.neg} := I_{y.ext.so} + I_{y.s.so} + I_{y.w} + I_{y.b} + I_{y.s.eff} + I_{y.d.eff} = 0.019\,m^4$

$A_{c.eff.loc} = 5.053\times10^4\,mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c\cdot f_y(t_d)\cdot A_{sd}}{\sigma_{com.Ed}\cdot\gamma_M} = -4.989\times10^{-2}\,mm^2$

$A_{sd.eff.neg} := A_{sb}$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed.

$A_{eff.loc} := \text{"OK" if } K_{A.2.1} = \text{"EJ OK"}; \rho_c\cdot A_{c.eff.loc.mod} + n_{sw.eff}\cdot t_d \text{ otherwise} = 432.173\,cm^2$

A_{su} Lower stiffener in tension --> no reduction

A_{sb} Tension --> No reduction

A_s No reduction if the side plate is in class 4.

A_{sw} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.113\times10^{-3}\,cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c\cdot\frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot t_d & \end{cases} = 7.916\,mm$

$y_{tot} := \frac{\begin{aligned} &n_{sw}\cdot h_{sw.eff}\cdot t_{sw}\frac{t_{d.eff}}{t_d}\left(t_d + \frac{h_{sw}}{2}\right) \ldots \\ &+ t_{sw.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{sw}\cdot t_{sw.eff}\cdot t_{d.eff}\left(\frac{t_d}{2}\right) \ldots \\ &+ t_s\cdot h_{s.eff.c1}\left(t_d + \frac{h_{s.eff.c1}}{2}\right) + t_s\cdot h_{s.eff.c2}\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) \ldots \\ &+ t_s\cdot h_{s1}\left(t_d + h_s - \frac{h_{s1}}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots \\ &+ A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sdy}\right) - \frac{h_{sw.y}}{2}\right] + A_b\left(h_w - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 428.793\,mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{sw}\cdot t_{sw.eff}\cdot t_{d.eff}^3}{12} + t_d\cdot n_{sw}\cdot t_{sw.eff}\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.774\times10^{-3}\,m^4$

$t_{sw.fin} := \sin\left(21.12\frac{\pi}{180}\right)\cdot t_{sw.fl} = 7.206\,mm$

Stiffeners over/under:

$I_{y.st.su.fin} := n_{sw}\left[\frac{t_{sw}\cdot h_{sw.eff}^3}{12} + n_{sw}\cdot t_{sw}\cdot h_{sw.eff}\left(t_d + \frac{h_{sw.eff}}{2} - z_{tp.eff.neg}\right)^2\right] = 2.486\times10^{-3}\,m^4$

$I_{y.st.su.1.fin} := 6.1.506\cdot10^{-6}\,m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \ldots + 2\left[t_{su}\cdot h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{sdy}\right) - \frac{h_{sw.y}}{2} - z_{tp.eff.neg}\right]^2\right] \ldots + 2\left[t_{su}\cdot h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sdy}\right) - \frac{h_{sw.y}}{2} - z_{tp.eff.neg}\right]^2\right] \ldots + 2\left[t_{su}\cdot h_{su}\left[t_d + h_s + \left(h_{wy} - s_{sdy}\right) - \frac{h_{sw.y}}{2} - z_{tp.eff.neg}\right]^2\right] = 9.965\times10^{-4}\,m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w h_w \left[h_{ts} - \frac{h_b}{2} + \frac{1}{2}(h_{wy}) - z_{tp.eff.neg} \right]^2 = 3.378 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b t_b^3}{12} + t_b b_b \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2} \right)^2 = 4.563 \times 10^{-3} m^4$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s h_{s.eff.c1}^3}{12} + t_s h_{s.eff.c1} \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg} \right)^2 + \\ \frac{t_s h_{s.eff.c2}^3}{12} + t_s h_{s.eff.c2} \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg} \right)^2 + \\ \frac{t_s h_{s.t}^2}{12} + t_s h_{s.t} \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg} \right)^2 \end{array} \right] = 5.887 \times 10^{-4} m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.778 \times 10^{-10} mm^4$

Load

Load capacity:

$q_M := 103 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_d = 360.5 \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.46 \times 10^4 kN \cdot m$

$q_V := 147 \frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V \cdot b_d = 514.5 \frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.63 \times 10^3 kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.041 m^3$

$M_{Rd} := W_{y.eff} \frac{f_y(t_d)}{\gamma_{M1}} = 1.472 \times 10^4 kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.992$

Shear capacity

$\eta = 1.20$ SS-EN 1993-1-5 S.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeners

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$a_{st.z.3} := a_{st.z.2} + s_{st} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 1.324m$

Moment of inertia stiffeners z-direction

$\eta_1 = \min\left(\frac{a}{h_{w.eff}}, 2.1\right) = 2.119$ Angle of web

$I_{st.z} := 2 t_{st} h_{st} \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2 \right] = 8.18 \times 10^{-3} m^4$ (two stiffeners)

Distance to stiffeners from z-axis:

Distance from z-axis to middle of the web area which is 15t each side of the stiffeners:

$a_{st.z.1} := \frac{h_b}{2} + s_{st} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) + \frac{t_{st}}{2} = 0.525m$

$a_{st.z.2} := a_{st.z.1} + s_{st} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 0.935m$

$a_{st.z.3} := a_{st.z.2} + s_{st} \cdot \cos\left(\alpha_w \frac{\pi}{180}\right) = 1.346m$

Moment of inertia of 15st of the web z-direction

$I_{sw.z} := 2 h_{web} \cdot t_w \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2 \right] = 0.021 m^4$

$I_{st.z.tot} := I_{st.z} + I_{sw.z} = 0.029 m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \left(\frac{h_{w.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34 \left(\frac{h_{w.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.561$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \sqrt{k_{\tau 1}}} = 0.546$ EC1-3-S eq.5.6

Shear buckling coefficient, web with more than two longitudinal stiffeners

$\frac{a}{h_w} = 1.493$

SS-EN 1993-1-5 (A.5)

$k_{\tau 2} := \max\left[5.34 \left(\frac{h_w}{a} \right)^2 \sqrt[4]{\frac{I_{st.z.tot}}{\left[\min\left(t_s, t_w\right)^3 h_w \right]}}, \frac{2.1}{\min\left(t_s, t_w\right)} \sqrt[3]{\frac{I_{st.z.tot}}{h_w}} \right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4 \left(\frac{h_{web}}{a} \right)^2 + k_{\tau 2} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 h_{web}} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 h_{web}}} & \text{otherwise} \end{cases} = 1.499 \times 10^3$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\frac{h_w}{t_w} = 1.2$ EC1993-1-3-S ANM.2 5.1

$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \frac{\varepsilon(t_s)}{\eta} \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{\left(37.4 t_w \cdot \varepsilon(t_w)\right) \sqrt{k_\tau}}, \lambda_{w1} \right) = 0.546$ SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ SS-EN 1993-1-5 5.3 Tabell 5.1, stiff end.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M1} := 1.00$ SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w f_y(t_s) h_{web} t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.324 \times 10^3 kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.63 \times 10^3 kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.996$ (contribution from two webs)

$V_{b.Rd} \leq \frac{\eta f_y(t_s) h_{web} t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{tof.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 423.793 \cdot mm$

$z_{tuf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 534.007 \cdot mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tof.eff.neg} \left| z_{tof.eff.neg} \right| f_y(t_d)}{\gamma_{M0}} + \frac{A_{tuf.eff.neg} \left| z_{tuf.eff.neg} \right| f_y(t_b)}{\gamma_{M0}} = 7.869 \times 10^3 kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tof} := a \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \left(h_s + h_{wy} \right)^2 \cdot f_y(t_s)} \right] = 0.603m$

$c_{tuf} := a \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \left(h_s + h_{wy} \right)^2 \cdot f_y(t_s)} \right] = 0.575m$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{tof} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{tuf} \cdot \gamma_{M1} \cdot \gamma_n} = 303.43 kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}} \right)^2 \right] & \text{if } M_{Ed} < M_{f.Rd.neg} = 0\,kN \\ 0 & \text{otherwise} \end{cases}$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} = \frac{\eta f_y(t_s) h_{web} t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} = V_{bw.Rd} + V_{bf.Rd} = 2.324 \times 10^3 kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.996$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of ir

$I_{y.f.eff.fin} := \frac{s_{so.eff} \cdot t_d^3}{12} + s_{so} \cdot s_{so.eff} \cdot t_d^{\,eff} \left(\frac{t_d.eff}{2} \right)^2 + \ldots$

$z_{tp.f.eff.neg} := \frac{+ s_{so} \cdot s_{so} \cdot t_d.eff \left(\frac{t_d.eff}{2} \right) + A_b \left(h_{ts} - \frac{t_b}{2} \right)}{A_{eff.f.tot}} = 358.687 \cdot mm$

The flanges

$A_{eff.f.tot} := s$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.029 m^3$ $= t_d.eff + A_b = 432.579 \cdot cm^2$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \frac{f_y(t_d)}{\gamma_M} = 1.023 \times 10^4 kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.472 \times 10^4 kN \cdot m$

Moment capacity check when $q_M = 103 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.992$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 147 \frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.996$

The Reduced stress method

-Changed side plate height to 470 mm

Material

$f_{yD} := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 335MPa & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} = 1$

$E := 210GPa$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate:

$t_b := 16mm$ $b_b := 1000mm$ $A_b := b_b \cdot t_b = 0.016m^2$

Top plate

$t_d := 10mm$ $b_d := 3500mm$ $A_d := b_d \cdot t_d = 0.035m^2$

Side plates:

$t_s := 12mm$ $h_s := 470mm$ $A_s := h_s \cdot t_s = 5.64 \cdot 10^{-3}m^2$

Webs:

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w \cdot t_w = 0.016m^2$

$h_{w.i} := 1258mm$ $h_{w.y} := 482.8mm$

Top stiffeners

$t_{std} := 16mm$ $b_{std} := 140mm$ $A_{std} := h_{std} \cdot t_{std} = 2.24 \cdot 10^{-3}m^2$

$s_{std} := 350mm$ $n_{std} := 9$

Web stiffeners:

$t_{stw} := 12mm$ $b_{stw} := 120mm$ $A_{stw} := h_{stw} \cdot t_{stw} = 1.44 \cdot 10^{-3}m^2$

$s_{stw} := 440mm$ $n_{stw} := 3$

Whole bridge

$t_{totalgir} := 18m$ $h_t := \frac{h_b}{2} + h_{w.y} + h_s + t_d = 0.971m$ $s_b := 2m$

$I_{xx} := 0.018193m^4$ From grasshopper

$y_{gt} := 0.56934m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_t := 87\frac{kN}{m^2}$

$Q := q_t \cdot s_b = 304.5\frac{kN}{m}$

$M_{Ed} := \frac{Q \cdot t_{totalgir}^2}{8} = 1.233 \cdot 10^4\,kN\,m$

$V_{Ed} := \frac{Q \cdot t_{totalgir}}{2} = 2.74 \cdot 10^3\,kN$

Top plate:

$\sigma_{x.Ed.top} := \frac{M_{Ed}}{I_{xx}}\left(h_w - \frac{t_d}{2} - y_{gt}\right) = 268.743\,MPa$ $\sigma_{x.Ed.up} := 0$

$S_{dy} := \frac{t_d \cdot b_d}{n_{std}+2}\left(h_w - \frac{t_d}{2} - y_{gt}\right) = 1.261 \cdot 10^{-3}\,m^3$

$\tau_{Ed.up} := \frac{V_{Ed} \cdot S_{dy}}{I_{xx} \cdot \min(t_d; t_{std})} = 15.835\,MPa$

Side plates:

$\sigma_{x.Ed.up} := \frac{M_{Ed}}{I_{xx}}(h_w - t_d - y_{gt}) = 265.354\,MPa$

$\sigma_{z.Ed.up} := \frac{Q_t}{2} = 12.688\,MPa$

$S_{dy} := (h_w - t_d - y_{gt})\cdot t_s \cdot \frac{(h_w - t_d - y_{gt})}{2} + \frac{t_d \cdot b_d}{n_{std}+2}\left(h_w - \frac{t_d}{2} - y_{gt}\right) = 0.011\,m^3$
$+ \frac{n_{std}}{2} \cdot b_{std} \cdot t_{std}\left(h_w - t_d - \frac{h_{std}}{2} - y_{gt}\right)$

$\tau_{Ed.up} := \frac{V_{Ed} \cdot S_{dy}}{I_{xx} \cdot t_s} = 15.464\,MPa$

Governing slenderness from FE-software

$\sigma_{x.LINE} := 4.01\frac{MPa}{m^2}\left(h_w - t_d - y_{gt}\right) = 265.354\,MPa$ From a linear stress analysis in Sofistik

$\sigma_{cr} := \sigma_{x.LINE} \cdot \eta = 350.61\,MPa$ 4.2449

$I_{xx} := \frac{t_d \cdot b_d^3}{12} + t_d \cdot b_d\left(h_t - t_d - \frac{h_{tot}}{4} - y_{gt}\right)^2 = 1.643 \cdot 10^{-3}\,m^3$

$\sigma_{xk} := \frac{t_d \cdot b_d}{A_d + n_{std} \cdot A_{std}} \cdot 1.013\left(\frac{t_d}{2} - y_{gt}\right)$

$\sigma_{cr.LINE} := 250.1\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistk

$\lambda_{sp} := \frac{\sigma_{x.LINE}}{\sigma_{cr}} = 2.875$

$\lambda_p := \sqrt{\frac{\sigma_{cr}}{\sigma_{cr}}} = 0.593$ Slenderness for all parts

Top plate check

$\psi_{sp} := 1$

$\rho_{p.up} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{sp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.sp} := \frac{\pi^2 \cdot E \cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\,MPa$

$I_{y.sp} := \frac{b_d \cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205 \cdot 10^{-7}\,m^4$

$y_{sp.d.up} := \frac{A_d\frac{t_d}{2} + n_{std}A_{std}\left(t_d + \frac{h_{std}}{2}\right)}{A_d + n_{std}A_{std}} = 32.411\,mm$

$I_{sp.up} := \frac{b_d t_d^3}{12} + A_d\left(y_{sp.d.up} - \frac{t_d}{2}\right)^2 + \dots = 1.052 \cdot 10^{-8}\,mm^4$
$+ n_{std}\left[\frac{t_{std}h_{std}^3}{12} + A_{std}\left(t_d + \frac{h_{std}}{2} - y_{sp.d.up}\right)^2\right]$

$\gamma_{sp} := \frac{I_{sp.up}}{y_{sp}} = 328.143$

$k_\sigma := \frac{\gamma_{sp} \cdot h_w}{A_d} = 0.576$

$\alpha_{sp} := \min\left(\frac{s_b}{h_d}; 0.5\right) = 0.571$ $\sqrt{\gamma_{sp}} = 4.256$

$k_{\sigma.p.sp} := \begin{cases} \frac{2\left[\left(1+\alpha_{sp}^2\right)^2 + \gamma_{sp} - 1\right]}{\alpha_{sp}^2\cdot(\psi_{sp}+1)\cdot(1-k_\sigma)} & \text{if } \alpha_{sp} < \frac{1}{\sqrt{\gamma_{sp}}} = 639.127 \\ \frac{4\cdot(1+\sqrt{\gamma_{sp}})}{(\psi_{sp}+1)\cdot(1-k_\sigma)} & \text{if } \alpha_{sp} \ge \frac{1}{\sqrt{\gamma_{sp}}} \end{cases}$

$\sigma_{cr.sp.up} := k_{\sigma.p.sp} \cdot \sigma_{E.sp} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1.std.sp} := \frac{3-\psi_{sp}}{5-\psi_{sp}} \cdot s_{std} = 0.175m$

$b_{2.sup.sp} := \frac{2}{5-\psi_{sp}} \cdot s_{std} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{d.c.sp} := b_{1.std.sp} + b_{2.sup.sp} = 0.35m$

$y_{sp.d.t.sp} := \frac{t_d b_{d.c.sp}\frac{t_d}{2} + A_{std}\left(t_d + \frac{h_{std}}{2}\right)}{t_d b_{d.c.sp} + A_{std}} = 0.034m$ combined center of gravity for stiffener and contributing top plate

$I_{d.c.sp} := \frac{b_{d.c.sp}t_d^3}{12} + t_{d.c.sp}t_d\left(y_{sp.d.t.sp} - \frac{t_d}{2}\right)^2 + \frac{t_{std}h_{std}^3}{12} + A_{std}\left(t_d + \frac{h_{std}}{2} - y_{sp.d.t.sp}\right)^2$

Combined moment of inertia

$\sigma_{cr.d.sp} := \frac{\pi^2 \cdot E \cdot I_{d.c.sp}}{A_{d.c.sp} \cdot s_{std}^2} = 1.026 \cdot 10^3\,MPa$ critical stress for kolummike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.sp} = \sigma_{cr.d}$

$\sigma_{cr.c.sp} := \sigma_{cr.d.sp}$

Reduction factor for column-like behaviour

$t_{sp} := \sqrt{\frac{I_{d.c.sp}}{A_{d.c.sp}}} = 0.045m$

$s_{sp} := \min\left(\left(s_{sp.d.t.sp} - t_d \cdot t_s + \frac{h_{tot}}{2} - y_{sp.d.t.sp}\right); 0.046m\right) = 0.046m$

$\alpha_{e.sp} := 0.49 + \frac{0.09}{\frac{s_{sp}}{t_{sp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{sp} := 0.5\left[1 + \alpha_{e.sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.791$ [6.3.1.2]

$\chi_{sp} := \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)}; 1\right) = 0.762$

Interpolation

$t_{sp} := \min\left(\left(\frac{\sigma_{cr.sp.up}}{\sigma_{cr.c.sp}} - 1.0\right); 1\right) = 0$

$\rho_{c.sp} := \left|\begin{array}{l} \left[\rho_{p.sp} - \chi_{sp}\left(\chi_{sp}\left(2 - t_{sp}\right) - t_{sp}^2\right)\frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right)\chi_{sp}\left(2 - t_{sp}\right) + \chi_{sp}^2\right]\alpha_{sh}}{\gamma_{M1}}\right] \cdot t_{sp} = 0.762 \\ 1 \quad \text{if } \left[\left(\rho_{p.sp} - \chi_{sp}\right)\chi_{sp}\left(2 - t_{sp}\right) + \chi_{sp}^2\right]\frac{\alpha_{sh}}{\gamma_{M1}} \cdot t_{sp} > 1 \end{array}\right.$

Shear

$\eta_1 = 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta_1 & \text{if } \lambda_p > \frac{0.83}{\eta_1} = 1.2 \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_1} < \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p > 1.08 \end{cases}$

$\frac{\rho_{c.sp}\cdot\alpha_{sh}}{\gamma_{M1}} = 0.771$

Side plates check

x-direction

$\sigma_{max} := f_y(t_s)$

$\sigma_{s2} := \left[\frac{-\sigma_{max}}{(\psi_{sp})} \cdot t + \sigma_{max}\right] \cdot 1$

$\psi_{sp.s} := \frac{\sigma(h_w - t_d - h_s)}{\sigma(h_w - t_d)} = -0.201$

$\sigma_{x.sup} := \frac{M_{Ed}}{I_{xx}}(h_w - t_d - y_{gt}) = 265.354\,MPa$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{xx}}(h_w - t_d - h_s - y_{gt}) = -53.239\,MPa$

$\psi_{sp1.s} := \max(3 + \psi_{sp.s}; 0) = 2.799$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.sup}} = -0.201$

$\rho_{p.sp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(\psi_{sp1}+1)}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma.sp.s} := \begin{cases} 1 & \text{if } \psi_{sp.s} \ge 1 \\ \frac{8.2}{(1.05 + \psi_{sp.s})} & \text{if } 1 > \psi_{sp.s} > 0 \\ 7.81 & \text{if } \psi_{sp.s} = 0 \\ 7.81 - 6.29\cdot\psi_{sp.s} + 9.78\cdot\psi_{sp.s}^2 & \text{if } 0 > \psi_{sp.s} > -1 \\ 23.9 & \text{if } \psi_{sp.s} = -1 \\ 5.98\cdot(1 - \psi_{sp.s})^2 & \text{if } -1 > \psi_{sp.s} > -3 \\ 95.68 & \text{if } \psi_{sp.s} < -3 \end{cases} = 9.466$

$\sigma_{cr.sp.s} := \sigma_{E.sp.s} \cdot k_{\sigma.sp.s} = 1.171 \cdot 10^3\,MPa$

Column-like behaviour

$\sigma_{cr.c.sp.s} := \frac{\pi^2 \cdot E \cdot t_s^2}{12\cdot(1-\nu^2)\cdot h_s^2} = 6.833\,MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.717$

$\chi_{sp} := \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)}; 1\right) = 0.892$

Interpolation

$t_{sp.s} := \min\left(\left(\frac{\sigma_{cr.sp.s}}{\sigma_{cr.c.sp.s}} - 1.0\right); 1\right) = 1$

$\rho_{c.sp.s} := \left|\begin{array}{l} \left[\rho_{p.sp} - \chi_{sp}\left(\chi_{sp}\left(2 - t_{sp.s}\right) - t_{sp.s}^2\right)\frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right)\chi_{sp}\left(2 - t_{sp.s}\right) + \chi_{sp}^2\right]\alpha_{sh}}{\gamma_{M1}}\right] \cdot t_{sp.s} = 1 \\ 1 \quad \text{if } \left[\left(\rho_{p.sp} - \chi_{sp}\right)\chi_{sp}\left(2 - t_{sp.s}\right) + \chi_{sp}^2\right]\frac{\alpha_{sh}}{\gamma_{M1}} \cdot t_{sp.s} > 1 \end{array}\right.$

Z-direction

$\psi_{sp.z} = 1$

$\alpha_{sp.z} = 0.34$ $\chi_{sp.z} = 0.7$

$\phi_{sp.z} := 0.5\left[1 + \alpha_{sp.z}\left(\lambda_p - \chi_{sp.z}\right) + \lambda_p^2\right] = 0.779$

$\rho_{p.z.sp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{sp.z} + \sqrt{\phi_{sp.z}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma.sp.z} = 4$

$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12\cdot(1-\nu^2)\cdot h_s^2} = 6.833\,MPa$

$\sigma_{cr.sp.z} := k_{\sigma.sp.z} \cdot \sigma_{E.sp.z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12\cdot(1-\nu^2)\cdot s_b^2} = 123.727\,MPa$

Interpolation

$t_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.sp.z}}{\sigma_{cr.c.sp.z}} - 1.0\right); 1\right) = 0$

$\rho_{c.sp.z} := \left|\begin{array}{l} \left[\rho_{p.sp} - \chi_{sp}\left(\chi_{sp}\left(2 - t_{sp.z}\right) - t_{sp.z}^2\right)\frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right)\chi_{sp}\left(2 - t_{sp.z}\right) + \chi_{sp}^2\right]\alpha_{sh}}{\gamma_{M1}}\right] \cdot t_{sp.z} = 0.892 \\ 1 \quad \text{if } \left[\left(\rho_{p.sp} - \chi_{sp}\right)\chi_{sp}\left(2 - t_{sp.z}\right) + \chi_{sp}^2\right]\frac{\alpha_{sh}}{\gamma_{M1}} \cdot t_{sp.z} > 1 \end{array}\right.$

Top plate stiffeners - check

x-direction

$$\rho_{p,k,lpb} := \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{lpb,k} := \frac{\sigma_{lb}\left(b_w - y_g - t_{fc}\right)}{\sigma_{lt}\left(b_w - t_{fc}\right)} = 0.642$$

$$\sigma_{E,lpb,k} := \frac{\pi^2 \cdot E_c^2}{12\left(1 - \nu^2\right) \cdot b_{lpb}^2} = 2.479 \times 10^3 \cdot MPa$$

$$k_{\sigma,lpb,k} := \begin{cases} 1 & \text{if } \psi_{lpb,k} \geq 1 \\ \frac{8.2}{\left(1.05 + \psi_{lpb,k}\right)} & \text{if } 1 > \psi_{lpb,k} > 0 \\ 7.81 & \text{if } \psi_{lpb,k} = 0 \\ 7.81 - 6.29 \cdot \psi_{lpb,k} + 9.78 \cdot \psi_{lpb,k}^2 & \text{if } 0 > \psi_{lpb,k} > -1 \\ 23.9 & \text{if } \psi_{lpb,k} = -1 \\ 5.98\left(1 - \psi_{lpb,k}\right)^2 & \text{if } -1 > \psi_{lpb,k} > -3 \\ 95.68 & \text{if } \psi_{lpb,k} < -3 \end{cases} = 4.163$$

$$\sigma_{cr,lp,lpb,k} := k_{\sigma,lpb,k} \cdot \sigma_{E,lpb,k} = 1.032 \times 10^4 \cdot MPa$$

Column-like behaviour

$$\sigma_{cr,c,lpb,k} := \frac{\pi^2 \cdot E_c^2}{12\left(1 - \nu^2\right) \cdot b_{lpb}^2} = 12.147 \cdot MPa$$

$$\chi_{lpb} := \chi_{lp}$$

Interpolation

$$\xi_{lpb,k} := \min\left(\max\left(\frac{\sigma_{cr,lp,lpb,k}}{\sigma_{cr,c,lpb,k}} - 1.0\right), 1\right) = 1$$

$$\rho_{c,lpb,k} := \begin{cases} \left[\left(\rho_{p,lpb} - \chi_{lpb}\right)\xi_{lpb,k}\left(2 - \xi_{lpb,k}\right) - 1\right] \cdot \chi_{lpb} & \text{if } \frac{\left[\left(\rho_{p,lpb} - \chi_{lpb}\right)\xi_{lpb,k}\left(2 - \xi_{lpb,k}\right) - 1\right]\chi_{lpb}}{\gamma_{M1}} < -1 \\ -1 & \text{if } \frac{\left[\left(\rho_{p,lpb} - \chi_{lpb}\right)\xi_{lpb,k}\left(2 - \xi_{lpb,k}\right) - 1\right]\chi_{lpb}}{\gamma_{M1}} < -1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,lpb} - \chi_{lpb}\right)\xi_{lpb,k}\left(2 - \xi_{lpb,k}\right) + \chi_{lpb}\right]\alpha_{lt}}{\gamma_{M1}} > 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,tp}}{\frac{\rho_{c,lpb,k} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,tp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.992$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,lpb,k} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,lpb,k} \cdot f_y}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,lpb,k} \cdot f_y}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,lpb,k} \cdot f_y}{\gamma_{M1}}}\right) + 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.611$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,sps}}{\frac{\rho_{c,lpb,k} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,sps}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.563$$

EWM - Changed side plate height

This chapter covers the EWM when changing the side plate height to 550 mm

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $h_d := 3500mm$ $t_d := 10mm$

$h_{d.0.5} := 1750mm$

$s_{so} := \frac{h_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 550mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $h_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{tt} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 1.051 \times 10^3 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 10mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 1.025 \times 10^3 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d h_d$ $A_s := 2 t_s h_s$ $A_w := 2 t_w h_w$ $A_b := t_b h_b$

$A_{so} := n_{so} t_{so} h_{so}$ $A_{su} := n_{su} t_{su} h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.125 \cdot m^2$

$A_{tot.test} := 0.125m^2$ $\Delta = 0.0mm$ EC1993-1-1 tab. 3.1

$A_c := A_{so} + A_d = 0.055 \cdot m^2$

Change in center of gravity, z_{tg} from top to centre of gravity

$z_{tg} := \frac{A_d \frac{t_d}{2} + A_{so}\left(t_d + \frac{h_{so}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wx}}{2}\right) + \ldots}{+ A_b\left(t_d + h_s + h_{wy}\right) + A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]} = 439.663 \cdot mm$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \frac{t_{so} h_{so}^3}{12} + n_{so} t_{so} h_{so}\left(t_d + \frac{h_{so}}{2} - z_{tg}\right)^2 = 2.641 \times 10^{-3} \cdot m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2 n_{su} h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tg}\right]^2 \ldots = 1.411 \times 10^{-3} \cdot m^4$
$+ 2 n_{su} h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tg}\right]^2 \ldots$
$+ 2 n_{su} h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tg}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 h_w t_w\left[h_{tt} - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tg}\right]^2 = 4.832 \times 10^{-3} \cdot m^4$

Deck:

$I_{y.d} := \frac{h_d t_d^3}{12} + t_d h_d\left(\frac{t_d}{2} - z_{tg}\right)^2 = 6.613 \times 10^{-3} \cdot m^4$

Bottom flange:

$I_{y.b} := \frac{h_b t_b^3}{12} + h_b b_b\left(h_{tt} - z_{tg} - \frac{t_b}{2}\right)^2 = 5.821 \times 10^{-3} \cdot m^4$

Side plate:

$I_{y.s} := 2 \frac{t_s h_s^3}{12} + 2 t_s h_s\left(t_d + \frac{h_s}{2} - z_{tg}\right)^2 = 6.485 \times 10^{-4} \cdot m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.022 \cdot m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$ Highest possible stress(in bottom flange) is the yield stress.

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tg}} z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tg}} z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$k_{\sigma.4.2}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$k_{\sigma.4.2}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \varepsilon(\psi) \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max(3 + \psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2) Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 560 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.28$

$\text{Condition} := \begin{cases} 62 \cdot \varepsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$

$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$k_{\sigma.4.1}(\psi_s) = 10.339$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.617$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tg} = 120.337 \cdot mm$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 429.663 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 429.663 \cdot mm$

$h_{s.eff.c1} := 0.4 h_{s.eff.c} = 171.865 \cdot mm$

$h_{s.eff.c2} := 0.6 h_{s.eff.c} = 257.798 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 550 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} t_s = 1.031 \times 10^4 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) t_s = 1.32 \times 10^4 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \frac{E \cdot \pi^2 \cdot t_s^2}{12\left(1 - \nu^2\right) h_s^2} = 934.124 \cdot MPa$ EC1993-1-5 eqA.1

Stiffeners

$t_{st} := t_d = 10 \cdot mm$

$h_w := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.674$

$k_{\sigma.4.2ii}(\psi_{so}) = 0.57$ Higher stress close to support

Class4 := "Class4" if $\frac{h_{so}}{t_{so}} > 21 \cdot \epsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2ii}(\psi_{so})}$ = "Lower than class 4"
"Lower than class 4" otherwise

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2ii}(\psi_{so}), h_{so}, t_{so}) = 0.502$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_d) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so} \cdot b_{so} = 140\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot b_{so.eff} = 2.016 \times 10^{4}\ mm^2$

Deck

$\frac{h_{sn}}{t_d} = 35$ $42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ := "Class4" if $\frac{h_{sn}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4"
"Lower than class 4" otherwise

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\ m$ $\psi_d := \psi_{neg}(z_d, -t_d) = 1$

$h_{web} := h_{so} - t_{so} = 140\ mm$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, t_{s.so}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461\ mm$

Areas:

$A_{so} := n_{so} \cdot s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.31 \times 10^{4}\ mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.053 \times 10^{4}\ mm^2$

More than three longitudinal stiffeners -> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{5}\ mm^4$

$y_{tp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411\ mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \dots + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{8}\ mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143$ $\alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02\ m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035\ m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.c} := \begin{cases} \left[2 \cdot \left(1 + \alpha^2\right)^2 + \gamma - 1\right] & \text{if } \alpha \le \sqrt{\gamma} \\ \left[2 \cdot \left(1 + \alpha^2\right) + \gamma\right] & \text{otherwise} \end{cases} = 639.127$

$\lambda_{cp} := \sqrt{\frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

orthotropic plate:

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d = 5.375 \times 10^{4}\ mm^2$ 3-1-5 eq.A.1

Buckling χ_c for class 4 sections:

$\overline{\lambda}_{A.c} := \sqrt{\frac{A_{c.eff} \cdot f_y(t_d)}{A_p}} = 0.952$

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^{4}\ kN$

Column like behaviour

$y_{tp.sl.c} := \frac{n_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot h_{so} + n_{so} \cdot t_d} = 34.268\ mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_{so}^3}{12} + n_{so} \cdot t_{so} \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \dots + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{-7}\ mm^4$

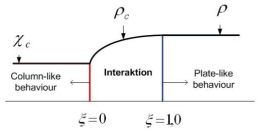
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{3}\ mm^2$

$A_{sl.eff} := t_{so} \cdot s_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{3}\ mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot s^2} = 1.026 \times 10^{3}\ MPa$

Slenderness ratio

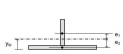
$\lambda_c := \sqrt{\frac{A_{sl.eff} \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.573$

calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\ mm$



$\overline{y}_c := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589\ mm$

$\alpha_c := \alpha - \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_c \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right) = 0$

ξ is zero -> column like behviour

Interaktion mellan de två olika beteendena(column/plate bucking):

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.601 \times 10^{4}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + s_{bo}} = -326.094\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{so})}{\gamma_M} = 281.032\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{so})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.032\ m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{s.eff} = 0.122\ m^2$

$z_{tp.eff.neg} := \frac{A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots + 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \dots + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{so.fy}\right) - \frac{h_{so.y}}{2}\right] \dots + A_b \cdot \left(h_n - \frac{h_b}{2}\right)}{A_{tot.eff}} = 449.805\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} \dots + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right]\right]\right] = 6.913 \times 10^{-4}\ m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d^2 \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 6.36 \times 10^{-3}\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.022\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{tf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^{4}\ mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.947 \times 10^{3}\ mm^2$

A_{so} Lower stiffener in tension -> no reduction

$A_{sf.eff.neg} := A_{so}$

A_b Tension -> No reduction Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the $A_{c.eff.loc.mod}$ which was checked before is not fulfilled, further reduction is needed and the area above is used.

A_{sw} Stiffener class 4 -> no reduction

Check: $\rho_c \cdot f_y(t_{so}) \le \frac{A_{c.eff.loc.mod}}{A_{sl}} \cdot f_y(t_d)$ $K_{A.2.1} = \text{"EJ OK"}$ $= 432.173\ cm^2$

Effective area, effective shared depth and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{so} + A_b = 1.132 \times 10^{3}\ cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d \end{cases} = 7.916\ mm$

$y_{eq.tot.neg} := \frac{n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{2} \cdot \left(t_d + \frac{h_{so}}{2}\right) \dots + h_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_d \cdot \left(\frac{t_d}{2}\right) \dots + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) \dots + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{so.fy}\right) - \frac{h_{so.y}}{2}\right] + A_b \cdot \left(h_n - \frac{h_b}{2}\right)}{A_{eff.tot}} = 466.066\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 6.834 \times 10^{-3}\ m^4$

$s_{so.eff} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{so0} = 7.206\ mm$

Stiffeners overlunder:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 3.038 \times 10^{-3}\ m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots + 2 \cdot t_{so} \cdot h_{so} \cdot \left[t_d + h_s + \frac{1}{3} \cdot \left(h_{wy} - s_{so.fy}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot t_{so} \cdot h_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{so.fy}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot t_{so} \cdot h_{so} \cdot \left[t_d + h_s + \left(h_{wy} - s_{so.fy}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 = 1.243 \times 10^{-3}\ m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\ m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[h_n - \frac{h_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 4.24 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot h_b^3}{12} + t_b \cdot h_b \cdot \left(h_n - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 5.322 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right]\right]\right] = 7.655 \times 10^{-4}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 2.144 \times 10^{10}\ mm^4$

Load

Load capacity:

$q_M := 115 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_d = 402.5 \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.63 \times 10^3 \text{ kN·m}$

$q_V := 160 \frac{kN}{m^2}$ The load capacity is q_v when checking the shear capacity.

$Q_V := q_V \cdot b_d = 560 \frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 5.04 \times 10^3 \text{ kN}$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{sp.eff.neg}} = 0.046 \text{ m}^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.633 \times 10^4 \text{ kN·m}$

$\frac{M_{Ed}}{M_{Rd}} = 0.998$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot t(t_{sq}) + t_w = 0.293 \text{ m}$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_h}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot s_{sw}}{2} = 0.503 \text{ m}$

$a_{st.z.2} := a_{st.z.1} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913 \text{ m}$

$a_{st.z.3} := a_{st.z.2} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324 \text{ m}$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 \cdot I_{sw} \cdot s_{sw} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3} \text{ m}^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$\overline{a_{web.z}} := \frac{h_h}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{sw}}{2} = 0.525 \text{ m}$

$\overline{a_{web.1}} := a_{st.z.1} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935 \text{ m}$

$\overline{a_{web.2}} := a_{st.z.2} + s_{sw} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346 \text{ m}$

Moment of inertia of 15st of the web z-direction

$I_{sw.z} := 2 \cdot h_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \text{ m}^4$

$I_{st.z.tot} := I_{st.z} + I_{sw.z} = 0.029 \text{ m}^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.643$

$k_{\tau 1d} := \max\left[9 \cdot \left(\frac{h_w}{a}\right) \cdot \sqrt[3]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_h, t_w\right)\right)^3 \cdot h_w}\right]^2}, \frac{2.1}{\min\left(t_h, t_w\right)} \cdot \sqrt[3]{\left[\frac{I_{st.z.tot}}{h_w}\right]^2}\right] = 4.782 \times 10^3$

Shear buckling coefficient, web with more than two longitudinal stiffeners

$k_\tau := \begin{cases} 5.34 + 4 \cdot \left(\frac{h_{web}}{a}\right)^2 + k_{\tau 1} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_h, t_w\right)^3 \cdot h_{web}} + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_h, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.386 \times 10^3$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\gamma_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_h, t_w\right)} \leq 31 \cdot \frac{\varepsilon(t_h)}{\eta} \cdot \sqrt{k_\tau} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.634$ SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$ SS-EN 1993-1-5 5.3 Tabell 5.1, stiff end.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_d) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1} \cdot \eta} = 2.521 \times 10^3 \text{ kN}$ SS-EN 1993-1-3 5.2 (5.2)

$V_{h.Rd} := V_{bw.Rd}$

$V_{Ed} = 5.04 \times 10^3 \text{ kN}$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 1$ (contribution from two webs)

$V_{h.Rd} \leq \frac{\eta \cdot f_y(t_d) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckding --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{tf.eff.neg} := z_{sp.eff.neg} - \frac{t_d}{2} = 461.066 \text{ mm}$

$a_{tf.eff.neg} := h_h - \frac{t_h}{2} - z_{sp.eff.neg} = 576.734 \text{ mm}$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \cdot \left|a_{tf.eff.neg}\right| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{tf.eff.neg} \cdot \left|a_{tf.eff.neg}\right| \cdot f_y(t_h)}{\gamma_{M0}} = 8.537 \times 10^3 \text{ kN·m}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_{fl} \cdot t_d^2 \cdot f_y(t_d)}{t_v \cdot \left(h_h + h_{wy}\right)^2 \cdot f_y(t_v)}\right] = 0.587 \text{ m}$

$c_{tf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_h \cdot t_h^2 \cdot f_y(t_h)}{t_v \cdot \left(h_h + h_{wy}\right)^2 \cdot f_y(t_v)}\right] = 0.564 \text{ m}$

$V_{bf.Rd.cored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{tf} \cdot \gamma_{M1} \cdot \eta} + \frac{b_h \cdot t_h^2 \cdot f_y(t_h)}{c_{tf} \cdot \gamma_{M1} \cdot \eta} = 310.52 \text{ kN}$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \text{ kN}$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_d) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{w.web} := V_{bw.Rd} + V_{bf.Rd} = 2.521 \times 10^3 \text{ kN}$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{h.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{h.Rd}} = 1$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.h.fin} = 0.012 \text{ m}^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot s_{so} \cdot t_{d.eff} + A_h = 432.579 \text{ cm}^2$

$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2} - \frac{t_{d.eff}}{2}\right) + n_{so} \cdot s_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2} + A_h \cdot \left(h_h - \frac{t_h}{2}\right)\right)}{A_{eff.f.tot}} = 388.277 \text{ mm}$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.031 \text{ m}^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.111 \times 10^4 \text{ kN·m}$

Results

$M_{pl.Rd} := M_{Rd} = 1.633 \times 10^4 \text{ kN·m}$

Moment capacity check when $q_M = 115 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.998$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 160 \frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 1$

The Reduced stress method

-Changed side plate height to 550 mm

Material

$f_{y0} := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ 335\,MPa & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$E = 210\,GPa$

$\nu = 0.3$ $G = \frac{E}{2\,(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate:

$t_b = 16\,mm$ $b_b = 1000\,mm$ $A_b = b_b t_b = 0.016\,m^2$

Top plate

$t_{tp} = 10\,mm$ $b_{tp} = 3500\,mm$ $A_{tp} = b_{tp} t_{tp} = 0.035\,m^2$

Side plates:

$t_s = 12\,mm$ $h_s = 550\,mm$ $A_s = h_s t_s = 6.6 \times 10^{-3}\,m^2$

Webs:

$t_w = 12\,mm$ $h_w = 1340\,mm$ $A_w = h_w t_w = 0.016\,m^2$

$h_{w.k} = 1258\,mm$ $h_{w.y} = 482.8\,mm$

Top stiffeners

$t_{st} = 16\,mm$ $b_{st} = 140\,mm$ $A_{st} = b_{st} t_{st} = 2.24 \times 10^{-3}\,m^2$

$s_{st} = 350\,mm$ $n_{st} = 9$

Web stiffeners

$t_{sw} = 12\,mm$ $b_{sw} = 120\,mm$ $A_{sw} = b_{sw} t_{sw} = 1.44 \times 10^{-3}\,m^2$

$s_{sw} = 440\,mm$ $n_{sw} = 3$

Whole bridge

$t_{bridge} = 18\,m$ $b_b = \frac{b_b}{2} + b_w + b_s + t_s = 1.051\,m$ $n_b = 2\,m$

$I_{yy} = 0.02194\,m^4$ From grasshopper

$y_{gt} = 0.61243\,m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q = 96\,\frac{kN}{m^2}$

$Q = q\,b_b = 336\,\frac{kN}{m}$

$M_{Ed} = \frac{Q\,t_{bridge}^2}{8} = 1.361 \times 10^4\,kN\,m$

$V_{Ed} = \frac{Q\,t_{bridge}}{2} = 3.024 \times 10^3\,kN$

Top plate:

$\sigma_{x.Ed.tp} = \frac{M_{Ed}}{I_{yy}}\left(h_b - y_{gt} - \frac{t_{tp}}{2} - y_g\right) = 269.43\,MPa$ $\sigma_{z.Ed.tp} = 0$

$y_g = \frac{t_b b_b}{A_{tot}}\left(h_w - \frac{t_b}{2} - y_g\right) = 1.383 \times 10^{-3}\,m^3$

$\tau_{Ed.tp} = \frac{V_{Ed}\,A_{tp}}{t_w\,\min(t_w, t_{tp})} = 15.876\,MPa$

Side plates

$\sigma_{x.Ed.sp} = \frac{M_{Ed}}{I_{yy}}\left(h_w - y_{gt} - y_g\right) = 266.33\,MPa$

$\sigma_{z.Ed.sp} = \frac{Q}{b_b} = 14\,MPa$

$A_{sp} = \left(h_w - t_{tp} - y_g\right) t_s + \frac{\left(h_w - t_{tp} - y_g\right)}{2} t_s + \frac{t_b b_b}{2}\left(h_w - \frac{t_b}{2} - y_g\right) = 0.012\,m^2$

$+ \frac{h_{sw}}{2} t_{sw} t_w \left(h_w - t_{tp} - \frac{h_{sw}}{2} - y_g\right)$

$\tau_{Ed.sp} = \frac{V_{Ed}\,A_{sp}}{I_{yy}\,n_w} = 15.539\,MPa$

Governing slenderness from FE-software

$\sigma_{x.USFI} = 3.69\,\frac{MN}{m^2}$ From a linear stress analysis in Sofistik

$\frac{Q}{n_b}\left(h_w - y_g\right) = 266.33\,MPa$

4.2449

$\sigma_y = \sigma_{x.USFI} = 354.24\,MPa$

$I_{sw} = \frac{t_s h_w^3}{12} + A_w\left(h_w - y_{gt} - \frac{h_w}{2}\right) = 1.804 \times 10^{-3}\,m^3$

$\alpha_{sh} = \frac{t_s h_w}{\sigma_y} = 1.002 \cdot \frac{N}{m^2} \cdot \frac{1}{m^2} \cdot \left(\frac{h_w}{2} - y_g\right)$

$\sigma_{cr.USFI} = 272.64\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} = \frac{\sigma_{cr.USFI}}{\sigma_y} = 2.84$

$\lambda_p = \sqrt{\frac{\sigma_y}{\sigma_{cr}}} = 0.594$ Slenderness for all parts

Top plate check

$\alpha_{tp} = 1$

$\rho_{p.tp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\,(3 + \alpha_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} = \frac{\pi^2\,E\,t_{tp}^2}{12\left(1 - \nu^2\right) b_{tp}^2} = 1.549\,MPa$

$I_{y.tp} = \frac{b_{tp} t_{tp}^3}{12\left(1 - \nu^2\right)} = 3.205 \times 10^{-7}\,m^4$

$y_{gt.tp} = \frac{A_{tp}\frac{t_{tp}}{2} + n_w A_w\left(t_{tp} + \frac{h_w}{2}\right)}{A_{tp} + n_w A_w} = 32.411\,mm$

$I_{y.tp} = \frac{b_{tp} t_{tp}^3}{12} + A_{tp}\left(y_{gt.tp} - \frac{t_{tp}}{2}\right)^2 + \ldots = 1.052 \times 10^{-8}\,mm^4$

$+ n_w\left[\frac{t_w h_w^3}{12} + A_w\left(t_{tp} + \frac{h_w}{2} - y_{gt.tp}\right)^2\right]$

$\gamma_{tp} = \frac{I_{yy}}{I_{y.tp}} = 328.143$

$k_{tp} = \frac{n_w A_w}{A_{tp}} = 0.576$

$n_{tp} = \max\left(\frac{n_s}{4}, 0.5\right) = 0.571$ $\sqrt{\gamma_{tp}} = 4.256$

$k_{\sigma.p.tp} = \begin{cases} \frac{2\left[\left(1 + \alpha_{tp}\sqrt{\gamma}\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\left(\gamma_{tp} - 1\right)\left(1 + k_{tp}\right)} & \text{if } \alpha_{tp}\sqrt{\gamma_{tp}} < 639.127 \\ \frac{4\left(1 + \sqrt{\gamma_{tp}}\right)}{\left(\gamma_{tp} + 1\right)\left(1 + k_{tp}\right)} & \text{if } \alpha_{tp}\sqrt{\gamma_{tp}} > 639.127 \end{cases}$

$\sigma_{cr.p.tp} = k_{\sigma.p.tp}\,\sigma_{E.tp} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1.eff.tp} = \frac{3 - \psi_{tp}}{5 - \psi_{tp}} b_{tp} = 0.175\,m$

$b_{2.eff.tp} = \frac{2}{5 - \psi_{tp}} b_{tp} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{el.1.tp} = b_{1.eff.tp} + b_{2.eff.tp} = 0.35\,m$

$A_{el.1.tp} = t_{tp} b_{el.1.tp} + A_{st} = 5.74 \times 10^{-3}\,m^2$

$y_{gt.el.1.tp} = \frac{t_{tp} b_{el.1.tp}\frac{t_{tp}}{2} + A_{st}\left(t_{tp} + \frac{h_{st}}{2}\right)}{t_{tp} b_{el.1.tp} + A_{st}} = 0.034\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{el.1.tp} = \frac{b_{el.1.tp} t_{tp}^3}{12} + b_{el.1.tp} t_{tp}\left(y_{gt.el.1.tp} - \frac{t_{tp}}{2}\right)^2 + \frac{t_{st} h_{st}^3}{12} + A_{st}\left(t_{tp} + \frac{h_{st}}{2} - y_{gt.el.1.tp}\right)^2$

Combined moment of inertia

$\sigma_{cr.el.tp} = \frac{\pi^2\,E\,I_{el.1.tp}}{A_{el.1.tp}\,b_{tp}^2} = 1.026 \times 10^3\,MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.el}$

$\sigma_{cr.c.tp} = \sigma_{cr.el.tp}$

Reduction factor for column-like behaviour

$i_{tp} = \sqrt{\frac{I_{el.1.tp}}{A_{el.1.tp}}} = 0.045\,m$

$s_{tp} = \min\left(b_{tp.el.1.tp} - t_{tp} - t_{st} + \frac{h_{st}}{2} - y_{gt.el.1.tp}\right) = 0.046\,m$

$\alpha_{c.tp} = 0.49 + \frac{0.09}{\frac{s_{tp}}{i_{tp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} = 0.5\left[1 + \alpha_{c.tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.791$ [6.3.1.2]

$\chi_{tp} = \min\left(\frac{1}{\left(\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}\right)}, 1\right) = 0.761$

Interpolation

$t_{tp} = \min\left(\max\left(\frac{\sigma_{cr.tp}}{\sigma_{cr.c.tp}} - 1.0\right), 1\right) = 0$

$\rho_{c.tp} = \begin{cases} \left[\left(\rho_{p.tp} - \chi_{tp}\right) b_{tp}\left(2 - t_{tp}\right) + \chi_{tp}^2\right] \cdot \frac{\left[\left(\rho_{p.tp} - \chi_{tp}\right) b_{tp}\left(2 - t_{tp}\right) + \chi_{tp}^2\right] n_{sh}}{\gamma_{M1}} & \text{if } \ldots < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p.tp} - \chi_{tp}\right) b_{tp}\left(2 - t_{tp}\right) + \chi_{tp}^2\right] n_{sh}}{\gamma_{M1}} > 1 \end{cases} = 0.761$

Shear

$\eta = 1.2$ 7.1 (2)

$\lambda_w = \begin{cases} \eta & \text{if } \lambda_p > \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} < \lambda_p < 1.08 \\ \frac{1.37}{\left(0.7 + \lambda_p\right)} & \text{if } \lambda_p > 1.08 \end{cases} = 1.2$

$\frac{\rho_{c.tp} n_{sh}}{\gamma_{M1}} = 0.763$

Side plates check

x-direction

$\sigma_{max} = \left(f_y\right)_0$

$\sigma_{t2} = \left[\frac{-\sigma_{min}}{\left(f_y\right)_0} + 1 + \sigma_{max}\right] - 1$

$\psi_{sp.x} = \frac{\sigma\left(h_w - y_g - h_s\right)}{\sigma\left(h_w - y_g\right)} = -0.28$

$\sigma_{x.top} = \frac{M_{Ed}}{I_{yy}}\left(h_w - y_g - y_{gt}\right) = 266.33\,MPa$

$\sigma_{x.bot} = \frac{M_{Ed}}{I_{yy}}\left(h_w - y_g - h_s - y_{gt}\right) = -74.676\,MPa$

$\psi_{sp1.x} = \max\left(3 \cdot \psi_{sp.x}, 0\right) = 2.72$ $\psi_{sp2} = \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.28$

$\rho_{p.sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\left(\psi_{sp1.x}\right)}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$\sigma_{E.sp.x} = \frac{\pi^2\,E\,t_s^2}{12\left(1 - \nu^2\right) b_{sp}^2} = 90.351\,MPa$

$\sigma_{cr.sp.x} = \sigma_{E.sp.x}\,k_{\sigma.sp.x} = 934.459\,MPa$

$k_{\sigma.sp.x} = \begin{cases} \frac{8.2}{\left(1.05 + \psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29\,\psi_{sp.x} + 9.78\,\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \end{cases} = 10.343$

Column-like behaviour $\psi = -1$

$\sigma_{cr.c.sp.x} = \frac{\pi^2\,E\,t_s^2}{12\left(1 - \nu^2\right) b_{sp}^2} \cdot \ldots^2 = 6.833\,MPa$ $\ldots = -3$

$\alpha_{sp} = 0.21$

$\phi_{sp} = 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.718$

$\chi_{sp} = \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)}, 1\right) = 0.892$

Interpolation

$t_{sp.x} = \min\left(\max\left(\frac{\sigma_{cr.sp.x}}{\sigma_{cr.c.sp.x}} - 1.0\right), 1\right) = 1$

$\rho_{c.sp.x} = \begin{cases} \left[\left(\rho_{p.sp} - \chi_{sp}\right) k_{sp.x}\left(2 - t_{sp.x}\right) + \chi_{sp}^2\right] \cdot \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right) k_{sp.x}\left(2 - t_{sp.x}\right) + \chi_{sp}^2\right] n_{sh}}{\gamma_{M1}} & \text{if } \ldots < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right) k_{sp.x}\left(2 - t_{sp.x}\right) + \chi_{sp}^2\right] n_{sh}}{\gamma_{M1}} > 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} = 1$

$\alpha_{sp.sp} = 0.34$ $\chi_{p0.sp} = 0.7$

$\phi_{sp.sp} = 0.5\left[1 + \alpha_{sp.sp}\left(\chi_p - \chi_{p0.sp}\right) + \chi_p\right] = 0.779$

$\rho_{p.sp} = \begin{cases} 1 & \text{if } \chi_p < 0.673 \\ \frac{1}{\left(\phi_{sp.sp} + \sqrt{\phi_{sp.sp}^2 - \chi_p^2}\right)} & \text{if } \chi_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma.p.sp.z} = 4$

$\sigma_{E.sp.z} = \frac{\pi^2\,E\,t_s^2}{12\left(1 - \nu^2\right) b_{sp}^2} = 6.833\,MPa$

$\sigma_{cr.sp.sp.z} = k_{\sigma.p.sp.z}\,\sigma_{E.sp.z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{cr.c.sp.z} = \frac{\pi^2\,E\,t_s^2}{12\left(1 - \nu^2\right) b_{sp}^2} = 90.351\,MPa$

Interpolation

$t_{sp.z} = \min\left(\max\left(\frac{\sigma_{cr.sp.sp.z}}{\sigma_{cr.c.sp.z}} - 1.0\right), 1\right) = 0$

$\rho_{c.sp.z} = \begin{cases} \left[\left(\rho_{p.sp} - \chi_{sp}\right) k_{sp.z}\left(2 - t_{sp.z}\right) + \chi_{sp}^2\right] \cdot \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right) k_{sp.z}\left(2 - t_{sp.z}\right) + \chi_{sp}^2\right] n_{sh}}{\gamma_{M1}} & \text{if } \ldots < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p.sp} - \chi_{sp}\right) k_{sp.z}\left(2 - t_{sp.z}\right) + \chi_{sp}^2\right] n_{sh}}{\gamma_{M1}} > 1 \end{cases} = 0.892$

Top plate stiffeners - check

x-direction

$\rho_{p.st.x} = \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$

$\psi_{sp.x} = \frac{\sigma\left(h_w - y_g - h_{sw}\right)}{\sigma\left(h_w - y_g\right)} = 0.674$

$\sigma_{E,cpt,x} := \frac{\pi^2 \cdot E \cdot t_{wb}^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_{wb}^2} = 2.479 \times 10^3 \cdot \text{MPa}$

$k_{\sigma,cpt,x} := \begin{cases} 1 & \text{if } \psi_{cpt,x} \geq 1 \\ \frac{8.2}{\left(1.05 + \psi_{cpt,x}\right)} & \text{if } 1 > \psi_{cpt,x} > 0 \\ 7.81 & \text{if } \psi_{cpt,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{cpt,x} + 9.78 \cdot \psi_{cpt,x}^2 & \text{if } 0 > \psi_{cpt,x} > -1 \\ 23.9 & \text{if } \psi_{cpt,x} = -1 \\ 5.98 \cdot \left(1 - \psi_{cpt,x}\right)^2 & \text{if } -1 > \psi_{cpt,x} > -3 \\ 95.68 & \text{if } \psi_{cpt,x} \leq -3 \end{cases} = 4.339$

$\sigma_{cr,cpt,cpt,x} := k_{\sigma,cpt,x} \cdot \sigma_{E,cpt,x} = 1.076 \times 10^4 \cdot \text{MPa}$

Column-like behaviour:

$\sigma_{ccc,cpt,x} := \frac{\pi^2 \cdot E \cdot t_{wb}^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_{wb}^2} = 12.147 \cdot \text{MPa}$

$\lambda_{cpt} := \lambda_{cp}$

Interpolation

$\xi_{cpt,x} := \min\left(\max\left(\frac{\sigma_{cr,cpt,cpt,x}}{\sigma_{ccc,cpt,x}} - 1.0\right), 1\right) = 1$

$\rho_{c,cpt,x} := \begin{cases} \left[\left(\rho_{0,cpt} - \lambda_{cpt}\right) \cdot \xi_{cpt,x} \cdot \left(2 - \xi_{cpt,x}\right)\right] \cdot \lambda_{cpt} & \text{if } \frac{\left[\left(\rho_{0,cpt} - \lambda_{cpt}\right) \cdot \xi_{cpt,x} \cdot \left(2 - \xi_{cpt,x}\right)\right] \cdot \sigma_{cm}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{0,cpt} - \lambda_{cpt}\right) \cdot \xi_{cpt,x} \cdot \left(2 - \xi_{cpt,x}\right) + \lambda_{cpt}\right] \cdot \sigma_{cm}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Top plate check

$\left(\frac{\sigma_{x,Ed,cp}}{\frac{\rho_{c,cpt,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,cp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.998$

Side plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 \cdot \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right) + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.615$

Top plate stiffeners check

$\left(\frac{\sigma_{x,Ed,sps}}{\frac{\rho_{c,sps,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sps}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.567$

EWM - Changed side plate height

This chapter covers the EWM when changing the side plate height to 630 mm

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

Side plate: $h_s := 630mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{tx} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 1.131 \times 10^3 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 10mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 1.105 \times 10^3 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d \cdot h_d$ **ustarameters:** $A_w := 2t_w \cdot h_w$ $A_b := h_b \cdot h_b$

$A_{so} := n_{so} \cdot a_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot a_{su} \cdot h_{su}$

E := 210GPa

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.127 \cdot m^2$ -1 tab 3.1

$A_s := A_{so} + A_{sl} = 0.055 \cdot m^2$ str

$\alpha(\psi) := \sqrt{\frac{235MPa}{\sigma_{max}}}$

Changed Center of gravity, z_{tp}

$$z_{tp} := \frac{\begin{pmatrix} A_d \frac{t_d}{2} + A_{so}\left(t_d + \frac{h_{so}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ + A_b\left(t_d + h_s + h_{wy}\right) + A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{wy}}{2}\right] \end{pmatrix}}{A_{tot}} = 477.842 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so}\left(\frac{t_{so} h_{so}^3}{12} + a_{so} t_{so} h_{so}\left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2\right) = 3.224 \times 10^{-3} \cdot m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2 t_{su} h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2 \dots = 1.702 \times 10^{-3} \cdot m^4$$
$$+ 2 t_{su} h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2 t_{su} h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 \cdot m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 t_w h_w\left[h_{tx} - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 5.862 \times 10^{-3} \cdot m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d\left(\frac{t_d}{2} - z_{tp}\right)^2 = 7.826 \times 10^{-3} \cdot m^4$

Bottom flange:

$I_{y.b} := \frac{b_b t_b^3}{12} + t_b b_b\left(h_{tx} - z_{tp} - \frac{t_b}{2}\right)^2 = 6.656 \times 10^{-3} \cdot m^4$

Side plate:

$I_{y.s} := 2\frac{t_s h_s^3}{12} + 2 t_s h_s\left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 8.533 \times 10^{-4} \cdot m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.026 \cdot m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} z - \sigma_{max}$

$\sigma_{1.neg}(\tau_1, \tau_2) := \max(\sigma_{neg}(\tau_1), \sigma_{neg}(\tau_2))$

$\sigma_{2.neg}(\tau_1, \tau_2) := \min(\sigma_{neg}(\tau_1), \sigma_{neg}(\tau_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} z + \sigma_{max}$

$\sigma_{1.pos}(\tau_1, \tau_2) := \max(\sigma_{pos}(\tau_1), \sigma_{pos}(\tau_2))$

$\sigma_{2.pos}(\tau_1, \tau_2) := \min(\sigma_{pos}(\tau_1), \sigma_{pos}(\tau_2))$

Buckling factor

$\psi_{neg}(\tau_1, \tau_2) := \frac{\sigma_{2.neg}(\tau_1, \tau_2)}{\sigma_{1.neg}(\tau_1, \tau_2)}$ $\psi_{pos}(\tau_1, \tau_2) := \frac{\sigma_{2.pos}(\tau_1, \tau_2)}{\sigma_{1.pos}(\tau_1, \tau_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2j}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(\psi)\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\max(3 + \psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 640 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.347$

$$Condition := \begin{cases} 62 \cdot \varepsilon(\psi_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(\psi_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 11.165$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.68$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 162.158 \cdot mm$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 467.842 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 467.842 \cdot mm$

$h_{s.eff.c1} := 0.4 \, h_{s.eff.c} = 187.137 \cdot mm$

$h_{s.eff.c2} := 0.6 \, h_{s.eff.c} = 280.705 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 630 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} t_s = 1.123 \times 10^4 \cdot mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{st}\right) t_s = 1.512 \times 10^4 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \frac{E \cdot \pi^2 \cdot s_{so}^2}{12\left(1 - \nu^2\right) h_s^2} = 768.848 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$t_{sti} := t_d = 10 \cdot mm$

$d_{sw} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.701$

$k_{\sigma.4.2k}(\psi_{so}) = 0.555$ Higher stress close to support

$Class4_{so} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon \cdot \sqrt{k_{\sigma.4.2k}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2k}(\psi_{so}), b_{so}, t_{so}) = 0.508$

$\rho_{4.so} := \rho_{4.2}(\lambda_{p.so}, \psi_{so}) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{4.so} \cdot b_{so} = 140\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot b_{so.eff} = 2.016 \times 10^{-6}\ m^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{sw} := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\ m \qquad \psi_{sd} := \psi_{neg}(z_d, z_d) = 1$

$n_{sww} := n_{so} - 1 = 5$ [tab. 5.2 ES1993-1-1] $k_{\sigma.sd} := k_{\sigma.4.1}(\psi_{sd}) = 4$

$\lambda_{p.sd} := \lambda_p(k_{\sigma.sd} \cdot s_{c.so} \cdot t_d) = 0.723$

$\rho_{sd} := \rho_{4.2}(\lambda_{p.sd}, \psi_{sd}) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_{sd} \cdot s_{c.so} = 321.461\ mm$

Areas:

$A_{sw} := n_{so} \cdot s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (b_{so.eff} + t_d) = 5.31 \times 10^{-4}\ m^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (b_{so.eff} + t_d) = 5.053 \times 10^{-4}\ m^2$

More than three longitudinal stiffners -> $k_{\sigma p}$ obtained with eq. A.2

$I_p := \frac{h_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5}\ m^4$

$y_{tp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + h_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot s_{so} \cdot h_{so} + h_d \cdot t_d} = 32.411\ mm$

$I_{sl} := \left[\frac{h_d \cdot t_d^3}{12} + h_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \dots + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{-6}\ m^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \qquad \alpha := \frac{a}{h_d} = 0.571$

$k_{\sigma.p} := \begin{cases} \frac{2\left[\left(1 + \alpha^2\right)^2 \cdot \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt{\gamma} \\ \frac{4\left(1 + \sqrt{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 639.127$

$A_{sl.eff.neg} = A_{sl}$ Bottom flange in tension

Check buckling stress, orthotropic plate:

$\sigma_{cr.p} := \frac{E \cdot \pi^2 \cdot I_p}{12 \cdot (1 - \nu^2) \cdot b_d^2} = 990.256\ MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{sw} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 5.375 \times 10^{-4}\ m^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^{4}\ kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268\ mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^3}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \dots + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{-7}\ m^4$

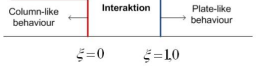
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{-3}\ m^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{-3}\ m^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.026 \times 10^{3}\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sd})}{\sigma_{cr.sl}}} = 0.573$

calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\ mm$

$\bar{e}_c := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589\ mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{\bar{e}_c}} = 0.522$

$\phi := 0.5\left[1 + \alpha_e \cdot \left(\lambda_c - 0.2\right) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaktion mellan de två olika beteendena(column/plate buckling):

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.612 \times 10^{4}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -328.404\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sd})}{\gamma_M} = 281.032\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{sd})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{so} \cdot s_{so} \cdot t_d = 0.032\ m^2$

$A_{tot.eff} := A_{sw.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{c.eff} = 0.124\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{sw.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ &\left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{st}}{2}\right)\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sdy}\right) - \frac{h_{so.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{tb} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 488.705\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right. + \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{sc} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2\right] \dots \left. + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{st}}{2} - z_{tp.eff.neg}\right)^2\right]\right] = 9.053 \times 10^{-4}\ m^4$

$I_{y.d} := n_{so} \cdot s_{so.eff} \cdot t_d^3 \cdot \left(\frac{\rho_c \cdot f_y(t_d) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M \cdot A_{c.eff.loc.mod}}\right)^2 \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 7.522 \times 10^{-3}\ m^4$

$I_{y.eff.sw} := I_{y.st.so} + y_{sw}^2 \cdot y_{sb} + y_{sa}^2 \cdot y_{s.eff} + I_{y.d.eff} = 0.027\ m^4$

Effektiv area in compression is calculated according to 4.5.1(4.5). If the effective area is greater than the gross cross section area, a reduction is needed and the area above is used.

Area

$A_{sw.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 432.173\ cm^2$

$A_{c.eff.loc} = 5.053 \times 10^{-4}\ m^2$

A_{so} Lower stiffener in tension -> no reduction

A_{sb} Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 -> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_{sw} + A_{so} + A_b = 1.151 \times 10^{-3}\ m^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.916\ mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot s_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) \dots \\ &+ s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_d \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) \dots \\ &+ t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ &+ A_{so} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sdy}\right) - \frac{h_{so.y}}{2}\right] + A_b \cdot \left(h_{tb} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 502.762\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so} \cdot s_{so.eff} \cdot t_{d.eff}^3}{12} + t_d \cdot n_{so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 7.965 \times 10^{-3}\ m^4$

$h_{sd.fin} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{sd} = 7.206\ mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 3.636 \times 10^{-3}\ m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots + 2 \cdot t_{su} \cdot h_{sd} \cdot \left[t_d + h_s + \frac{1}{2}\left(h_{wy} - s_{sdy}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot t_{su} \cdot h_{sd} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sdy}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot t_{su} \cdot h_{sd} \cdot \left[t_d + h_s + \left(h_{wy} - s_{sdy}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 = 1.525 \times 10^{-3}\ m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\ m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_b - \frac{t_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 5.235 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{tb} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 6.151 \times 10^{-3}\ m^4$

Side plate:

$$I_{y.x.fin} := 2\left[\frac{t_b\,h_{w.eff.c1}^{3}}{12} + t_b\,h_{w.eff.c1}\left(t_d + \frac{h_{w.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 + \cdots + \frac{t_b\,h_{w.eff.c2}^{3}}{12} + t_b\,h_{w.eff.c2}\left(t_d + h_{s.c} - \frac{h_{w.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 + \cdots + \frac{t_s\,h_{s.s}^{3}}{12} + t_s\,h_{s.s}\left(t_d + h_b - \frac{h_{s.s}}{2} - z_{tp.eff.neg}\right)^2\right] = 9.779 \times 10^{-4}\ m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 2.549 \times 10^{10}\ mm^4$$

Load

Load capacity:

$q_M := 127\ \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$$Q_M := q_M \cdot b_d = 444.5\ \frac{kN}{m}$$

$$M_{Ed} := \frac{Q_M \cdot l_{bridge}^{2}}{8} = 1.8 \times 10^{4}\ kN \cdot m$$

$q_V := 165\ \frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$$Q_V := q_V \cdot b_d = 577.5\ \frac{kN}{m}$$

$$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 5.197 \times 10^{3}\ kN$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.051\ m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.8 \times 10^{4}\ kN \cdot m$$

$\frac{M_{Ed}}{M_{Rd}} = 1$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeners

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$$h_{web} := 2 \cdot 15 \cdot t(t_w) \cdot t_w = 0.293\ m$$

$$\alpha_w := \min\left(\frac{h_{ws}}{h_w}, \frac{180}{\pi}\right) = 21.119$$ Angle of web

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{h_b}{2} + s_{wab} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ss}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{ss}}{2} = 0.503\ m$$

$$a_{st.z.2} := a_{st.z.1} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\ m$$

$$a_{st.z.3} := a_{st.z.2} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\ m$$

Moment of inertia stiffeners z-direction (two stiffeners)

$$I_{st.z} := 2 \cdot t_{ss} \cdot h_{ss}\left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\ m^4$$

Distance from z-axis to middle of the web area which is 15ct each side of the stiffeners:

$$a_{wmid} := \frac{h_b}{2} + s_{wab} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ss}}{2} = 0.525\ m$$

$$a_{smid} := a_{st.z.1} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\ m$$

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00\left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34\left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.737$$

$$I_{st.z} = 1.021 \cdot m^4$$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s)\sqrt{k_{\tau 1}}} = 0.72$$ EC1-3-5 eq.5.6

Shear buckling coefficient, side plate

Shear buckling coefficient; web with more than two longitudinal stiffeners

$$a_w := \frac{a}{h_w} = 1.493$$

SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[\left(\frac{h_w}{a}\right)^2 \sqrt{\frac{I_{st.z.tot}}{\min\left(t_s \cdot h_w\right)^3 \cdot h_w}}, \frac{2.1}{\min\left(t_s \cdot t_w\right)} \sqrt{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^{3}$$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18\frac{I_{st.z.tot}}{\min\left(t_s \cdot t_w\right)^3 \cdot h_{web}} + 2.2\sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s \cdot t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \\ \frac{4.1 + \alpha^2}{\alpha^2} & \end{cases} = 1.288 \times 10^{3}$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\overline{\lambda_w} := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s \cdot t_w\right)} \leq 31 \cdot \frac{\varepsilon(t_s)}{\eta}\sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{\left(37.4 \cdot a_w \cdot \varepsilon(t_w)\sqrt{k_\tau}\right)}, \lambda_{w1}\right) = 0.72$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.152$$ SS-EN 1993-1-5 5.3 Tabell 5.1, stiff end.

Kapacity of the web

$\overline{\gamma_{M0}} := 1.00$ $\overline{\gamma_m} := 1.2$ Safety class 3

$\overline{\gamma_{M1}} := 1.00$ SB3 "bärande konstruktioner"

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_\eta} = 2.609 \times 10^{3}\ kN$$ SS-EN 1993-1-3 5.2 (5.2)

$$V_{b.Rd} := V_{bw.Rd}$$

$$V_{Ed} = 5.197 \times 10^{3}\ kN$$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.996$ (contribution from two webs)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$$a_{tf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 497.762\ mm$$

$$a_{uf.eff.neg} := h_b - \frac{t_b}{2} - z_{tp.eff.neg} = 620.038\ mm$$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area:

$$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \cdot \left|a_{tf.eff.neg}\right| \cdot f_y(t_d)}{\gamma_{M0}} + \cdots + \frac{A_{uf.eff.neg} \cdot \left|a_{uf.eff.neg}\right| \cdot f_y(t_b)}{\gamma_{M0}} = 9.202 \times 10^{3}\ kN \cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$$M_{Ed} < M_{f.Rd.neg} = 0$$

$$c_{tf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot \left(h_w + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.575\ m$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot \left(h_w + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.555\ m$$

$$V_{bf.Rd.cored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{tf} \cdot \gamma_{M1} \cdot \gamma_\eta} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_\eta} = 316.381\ kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\ kN$$

Shear capacity stiffened web:

$\frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{bw.Rd} + V_{bf.Rd} = 2.609 \times 10^{3}\ kN$

Check:

$\frac{V_{Ed}}{2} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.996$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.014\ m^4$$

The flanges effective area:

$$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{su} \cdot s_{su.eff} \cdot t_{d.eff} + n_{su} \cdot t_{su} \cdot t_{d.eff} + A_b = 432.579\ cm^2$$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + \cdots + n_{su} \cdot s_{su} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_w - \frac{h_b}{2}\right)}{A_{eff.f.tot}} = 417.867\ mm$$

$$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.034\ m^3$$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.199 \times 10^{4}\ kN \cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 1.8 \times 10^{4}\ kN \cdot m$$

Moment capacity check when $q_M = 127\ \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 1$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 165\ \frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.996$

The Reduced stress method

-Changed side plate height to 630 mm

Material

$f_{y0} := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ 335\,MPa & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$k := 210\,GPa$

$\nu = 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$t_b := 16mm$ $b_b := 1000mm$ $A_b := b_b t_b = 0.016m^2$

Top plate

$t_p := 10mm$ $b_p := 3500mm$ $A_p := b_p t_p = 0.035m^2$

Side plates

$t_s := 12mm$ $h_s := 630mm$ $A_s := h_s t_s = 7.56\cdot 10^{-3}m^2$

Webs

$t_w := 12mm$ $h_w := 1340mm$ $A_w := h_w t_w = 0.016m^2$

$h_{w,n} := 1250mm$ $h_{w,p} := 482.8mm$

Top stiffeners

$t_{sti} := 16mm$ $h_{sti} := 140mm$ $A_{sti} := h_{sti} t_{sti} = 2.24\cdot 10^{-3}m^2$

$s_{sti} := 350mm$ $n_{sti} := 9$

Web stiffeners

$t_{wsti} := 12mm$ $h_{wsti} := 120mm$ $A_{wsti} := h_{wsti} t_{wsti} = 1.44\cdot 10^{-3}m^2$

$s_{wsti} := 440mm$ $n_{wsti} := 3$

Whole bridge

$l_{bridge} := 18m$ $y_{s1} := \frac{h_s}{2} + h_{w,p} + t_b + t_p = 1.131m$ $y_{s2} := 2m$

$I_{sti} := 0.026102m^4$ From grasshopper

$y_{fg} := 0.6539m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 105\frac{kN}{m^2}$

$Q := q\cdot b_p = 367.5\frac{kN}{m}$

$M_{Ed} := \frac{Q\cdot l_{bridge}^2}{8} = 1.488\cdot 10^4\,kN\,m$

$V_{Ed} := \frac{Q\cdot l_{bridge}}{2} = 3.308\cdot 10^3\,kN$

Top plate

$\sigma_{x,Ed,tp} := \frac{M_{Ed}}{I_{eff}}\left(y_{s1} - y_{fg} - \frac{t_p}{2} - y_{fg}\right) = 269.546\,MPa$ $\sigma_{z,Ed,tp} := 0$

$S_{fg} := \frac{t_p t_p}{n_{sti}+2}\left(y_{s1} - \frac{t_p}{2} - y_{fg}\right) = 1.504\cdot 10^{-3}m^3$

$\tau_{Ed,tp} := \frac{V_{Ed} S_{fg}}{I_{eff}\cdot\min(t_p; t_{sti})} = 15.882\,MPa$

Side plates

$\sigma_{x,Ed,sp} := \frac{M_{Ed}}{I_{eff}}\left(y_{s1} - t_p - y_{fg}\right) = 266.695\,MPa$

$\sigma_{z,Ed,sp} := \frac{Q}{A_s} = 15.313\,MPa$

$S_{fg} := \left(y_{s1} - t_p - y_{fg}\right)\cdot t_s\cdot\frac{\left(y_{s1} - t_p - y_{fg}\right)}{2} + \frac{t_p t_p}{n_{sti}+2}\left(y_{s1} - \frac{t_p}{2} - y_{fg}\right) + \frac{n_{sti}}{n_{sti}+2} b_{sti} t_{sti}\left(y_{s1} - t_p - \frac{h_{sti}}{2} - y_{fg}\right) = 0.014\,m^3$

$\tau_{Ed,sp} := \frac{V_{Ed} S_{fg}}{I_{eff}\cdot 2 t_s} = 15.575\,MPa$

Governing slenderness from FE-software

$\sigma_{x,1,Ed} := 3.42\frac{MPa}{\frac{kN}{m^2}} 71.772\,MPa$ From a linear stress analysis in Sofistik

$\sigma_{fg} := \sigma_{x,1,Ed}\cdot q = 359.1\,MPa$ $\lambda_{fg} := \sqrt{\frac{\sigma_{fg}}{\sigma_{cr}}} = 0.894$ $\sigma_{cr} := \frac{f_y(y_{fg})}{\sigma_{fg}} = 0.989$ $\frac{I_{eff}}{S_{fg}} = 66.503\,m$

$\gamma_{sp} := \frac{t_s^3 t_s}{n_{sti}+2}\left(y_{s1} - t_p - \frac{h_{sti}}{2} - y_{fg}\right)^2 + \frac{t_p t_p^3}{n_{sti}+2}\left(y_{s1} - \frac{t_p}{2} - y_{fg}\right)^2 = 1.967\cdot 10^{-3}m^3$

$\sigma_{cr,1,Ed} := 294.64\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\sigma_{cr,1,Ed}}{q} = 2.806$

$\lambda_p := \sqrt{\frac{f_{y0}}{\sigma_{cr}}} = 0.594$ Slenderness for all parts

Top plate check

$\psi_{fg} := 1$

$\rho_{p,tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{fg})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E,tp} := \frac{\pi^2 k_\sigma}{12\cdot(1-\nu^2)}\left(\frac{t_p}{b_p}\right)^2 = 1.549\,MPa$

$I_{p,tp} := \frac{b_p t_p^3}{12\cdot(1-\nu^2)} = 3.205\cdot 10^{-7}m^4$

$y_{p,d,tp} := \frac{A_p\frac{t_p}{2} + A_{sti} n_{sti}\left(t_p + \frac{h_{sti}}{2}\right)}{A_p + n_{sti} A_{sti}} = 32.411\,mm$

$I_{y,tp} := \frac{b_p t_p^3}{12} + A_p\left(y_{p,d,tp} - \frac{t_p}{2}\right)^2 + n_{sti}\left[\frac{t_{sti} h_{sti}^3}{12} + A_{sti}\left(t_p + \frac{h_{sti}}{2} - y_{p,d,tp}\right)^2\right] = 1.052\cdot 10^{-6}m^4$

$n_{sti} = 9$

$\gamma_{tp} := \frac{I_{y,tp}}{y_{fg}} = 328.143$

$k_{fg} := \frac{n_{sti} A_{sti}}{A_p} = 0.576$

$\alpha_{fg} := \min\left(\frac{\gamma_{sp}}{\gamma_{tp}}; 0.5\right) = 0.571$ $\sqrt{\frac{a}{t_p}} = 4.256$

$k_{\sigma,p,tp} := \begin{cases} \frac{2\left[\left(1+\alpha_{fg}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{fg}^2\left(\psi_{fg}-1\right)\left(1+k_{fg}\right)} & \text{if } \alpha_{fg}\sqrt{\frac{a}{t_p}} = 639.127 \\ \frac{4\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{fg}-1\right)\left(1+k_{fg}\right)} & \text{if } \alpha_{fg}\sqrt{\frac{a}{t_p}} \end{cases}$

$\sigma_{cr,p,tp} := k_{\sigma,p,tp}\cdot\sigma_{E,tp} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$h_{1,sti,tp} := \frac{3-\psi_{fg}}{5+\psi_{fg}} b_{sti} = 0.175m$

$b_{2,sup,tp} := \frac{2}{5-\psi_{fg}} b_{sti} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{eff,1,tp} := h_{1,sti,tp} + b_{2,sup,tp} = 0.35m$

$y_{fg,d,1,tp} := \frac{t_p b_{eff,1,tp}\frac{t_p}{2} + A_{sti}\left(t_p + \frac{h_{sti}}{2}\right)}{t_p b_{eff,1,tp} + A_{sti}} = 0.034m$ Combined center of gravity for stiffener and contributing top plate.

$I_{y,1,tp} := \frac{b_{eff,1,tp} t_p^3}{12} + b_{eff,1,tp} t_p\left(y_{fg,d,1,tp} - \frac{t_p}{2}\right)^2 + \frac{t_{sti} h_{sti}^3}{12} + A_{sti}\left(t_p + \frac{h_{sti}}{2} - y_{fg,d,1,tp}\right)^2$ Combined moment of inertia

$\sigma_{cr,d,tp} := \frac{\pi^2 k_{1,1,tp}}{A_{sti,1,tp}\frac{l_{bridge}^2}{n_{sti}}} = 1.026\cdot 10^3\,MPa$ Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,p} = \sigma_{cr,d}$

$\sigma_{cr,c,tp} := \sigma_{cr,d,tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{y,1,tp}}{A_{sti,1,tp}}} = 0.045m$

$e_{tp} := \max\left(\left(y_{fg,d,1,tp} - y_{fg} + \frac{h_{sti}}{2}\right); -y_{fg,d,1,tp}\right) = 0.046m$

$\alpha_{e,tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e,tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.791$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\left(\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}\right)}; 1\right) = 0.761$

Interpolation

$t_{tp} := \max\left(\left(\frac{\sigma_{cr,tp}}{\sigma_{cr,c,tp}} - 1.0\right); 1\right) = 0$

$\rho_{c,tp} := \begin{cases} \left[\left(\rho_{p,tp} - \chi_{tp}\right) k_{tp}\left(2 - t_{tp}\right) + \chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right) k_{tp}\left(2 - t_{tp}\right) + \chi_{tp}\right] n_{sti}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp} - \chi_{tp}\right) k_{tp}\left(2 - t_{tp}\right) + \chi_{tp}\right] n_{sti}}{\gamma_{M1}} \ge 1 \end{cases} = 0.761$

Shear

$\eta_1 = 1.2$ 7.1 (2)

$\lambda_w := \begin{cases} \eta_1 & \text{if } \lambda_p < \frac{0.83}{\eta_1} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_1} \le \lambda_p < 1.08 \\ \frac{1.37}{\left(0.7 + \lambda_p\right)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$

$\frac{\sigma_{x,tp}\cdot\alpha_{fg}}{\gamma_{M1}} = 0.753$

Side plates check

x-direction

$\sigma_{max} := f_y(y_{fg})$

$\sigma(z) := \left[\frac{\sigma_{max}}{\left(t_p - y_{fg}\right)}\cdot z + \sigma_{max}\right] - 1$

$\psi_{sp,x} := \frac{\sigma\left(h_s - t_p - y_{fg}\right)}{\sigma\left(h_s - t_p - y_{fg}\right)} = -0.347$

$\sigma_{x,sup} := \frac{M_{Ed}}{I_{eff}}\left(h_s - t_p - y_{fg}\right) = 266.695\,MPa$

$\sigma_{x,bot} := \frac{M_{Ed}}{I_{eff}}\left(h_s - t_p - h_s - y_{fg}\right) = -92.54\,MPa$

$\psi_{sp1,x} := \max\left(3 + \psi_{sp,x}; 0\right) = 2.653$ $\psi_{sp1} := \frac{\sigma_{x,bot}}{\sigma_{x,sup}} = -0.347$

$\rho_{p,x,sp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\cdot(\psi_{sp1,x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,sp,x} := \begin{cases} 8.2 & \text{if } \psi_{sp,x} \ge 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp,x}\right)} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\,\psi_{sp,x} + 9.78\,\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98\cdot\left(1 - \psi_{sp,x}\right)^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 95.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 11.17$

$\sigma_{cr,p,sp,x} := \sigma_{E,sp,x}\cdot k_{\sigma,sp,x} = 769.191\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,x} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot h_s^2} = 6.833\,MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.717$

$\chi_{sp} := \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)}; 1\right) = 0.892$

Interpolation

$t_{sp,x} := \max\left(\left(\frac{\sigma_{cr,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0\right); 1\right) = 1$

$\rho_{c,sp,x} := \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right) k_{sp,x}\left(2 - t_{sp,x}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right) k_{sp,x}\left(2 - t_{sp,x}\right) + \chi_{sp}\right] n_{sti}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right) k_{sp,x}\left(2 - t_{sp,x}\right) + \chi_{sp}\right] n_{sti}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

Z-direction

$\psi_{sp,z} = 1$

$\alpha_{p,sp} = 0.34$ $\chi_{p,sp} = 0.7$

$\phi_{p,sp} := 0.5\left[1 + \alpha_{p,sp}\left(\lambda_p - \chi_{p,sp}\right) + \lambda_p^2\right] = 0.779$

$\rho_{p,sp,z} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{p,sp} + \sqrt{\phi_{p,sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,p,sp,z} = 4$

$\sigma_{E,sp,z} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot a^2} = 6.833\,MPa$

$\sigma_{cr,p,sp,z} := k_{\sigma,p,sp,z}\cdot\sigma_{E,sp,z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{cr,c,sp,z} := \frac{\pi^2 E t_s^2}{12\cdot(1-\nu^2)\cdot h_s^2} = 68.862\,MPa$

Interpolation

$t_{sp,z} := \max\left(\left(\frac{\sigma_{cr,p,sp,z}}{\sigma_{cr,c,sp,z}} - 1.0\right); 1\right) = 0$

$\rho_{c,sp,z} := \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right) k_{sp,z}\left(2 - t_{sp,z}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right) k_{sp,z}\left(2 - t_{sp,z}\right) + \chi_{sp}\right] n_{sti}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right) k_{sp,z}\left(2 - t_{sp,z}\right) + \chi_{sp}\right] n_{sti}}{\gamma_{M1}} \ge 1 \end{cases} = 0.892$

Top plate stiffeners - check

x-direction

$\rho_{p,x,sp} := \begin{cases} 1 & \text{if } \lambda_{p,x} \le 0.748 \\ \frac{\lambda_{p,x} - 0.188}{\lambda_{p,x}^2} & \text{if } \lambda_{p,x} > 0.748 \end{cases} = 1$

$\psi_{sp,x,s} := \frac{\sigma 1 b_{sp,x} - \sigma_{sp} \cdot h_{sp}}{\sigma 1 b_{sp,x} - \sigma_{sp}} = 0.701$

$\sigma_{E,sp,x,s} := \frac{\pi^2 \cdot E \cdot t_{sp,x}^2}{12 \cdot (1 - \nu^2) \cdot b_{sp,x}^2} = 2.479 \times 10^3 \cdot MPa$

$k_{sp,sp,x,s} := \begin{cases} 1 & \text{if } \psi_{sp,x,s} \ge 1 \\ \frac{8.2}{(1.05 + \psi_{sp,x,s})} & \text{if } 1 > \psi_{sp,x,s} > 0 \\ 7.81 & \text{if } \psi_{sp,x,s} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp,x,s} + 9.78 \cdot \psi_{sp,x,s}^2 & \text{if } 0 > \psi_{sp,x,s} > -1 \\ 23.9 & \text{if } \psi_{sp,x,s} = -1 \\ 5.98 \cdot (1 - \psi_{sp,x,s})^2 & \text{if } -1 > \psi_{sp,x,s} > -3 \\ 95.68 & \text{if } \psi_{sp,x,s} < -3 \end{cases} = 4.58$

$\sigma_{cr,sp,sp,x,s} := k_{sp,sp,x,s} \cdot \sigma_{E,sp,x,s} = 1.135 \times 10^4 \cdot MPa$

Column-like behaviour

$\sigma_{cr,c,sp,x,s} := \frac{\pi^2 \cdot E \cdot t_{sp}^2}{12 \cdot (1 - \nu^2) \cdot b_{sp}^2} = 12.147 \cdot MPa$

$\chi_{sp,x} := \chi_{sp}$

Interpolation

$\xi_{sp,x,s} := \min\left(\max\left(\frac{\sigma_{cr,sp,sp,x,s}}{\sigma_{cr,c,sp,x,s}} - 1.0, 1\right), 1\right) = 1$

$\rho_{cr,sp,x,s} := \begin{cases} \left[\left(\rho_{sp,sp,x} - \chi_{sp}\right) \cdot k_{sp,x,s} \cdot \left(2 - \xi_{sp,x,s}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{sp,sp,x} - \chi_{sp}\right) \cdot k_{sp,x,s} \cdot \left(2 - \xi_{sp,x,s}\right) + \chi_{sp}\right] \cdot \sigma_{sp}}{\gamma_{M1}} \cdot 1 = 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{sp,sp,x} - \chi_{sp}\right) \cdot k_{sp,x,s} \cdot \left(2 - \xi_{sp,x,s}\right) + \chi_{sp}\right] \cdot \sigma_{sp}}{\gamma_{M1}} = 1 \end{cases} = 1$

Top plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{cr,sp,x,s} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.998$

Side plate check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{cr,sp,x,s} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{cr,sp,x,s} \cdot f_y}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{cr,sp,x,s} \cdot f_y}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{cr,sp,x,s} \cdot f_y}{\gamma_{M1}}}\right) = 0.616$

$+ 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2$

Top plate stiffeners check

$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{cr,sp,x,s} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.568$

EWM - Changed side plate height

This chapter covers the EWM when changing the side plate heigh to 710 mm

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $h_d := 3500mm$ $t_d := 10mm$

$h_{d.0.5} := 1750mm$

$s_{so} := \frac{h_d}{n_{s.so}} = 350\cdot mm$

Side plate: $h_s := 710mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $h_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su}\cdot\cos\left(21.12\cdot\frac{\pi}{180}\right) = 111.939\cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 1.211\times 10^{3}\cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 1.185\times 10^{3}\cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}}\cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}}\cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1-\psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

Area

$A_d := t_d\cdot h_{uarameter}$ $A_s := 2\cdot t_s\cdot h_s$ $A_w := 2\cdot t_w\cdot h_w$ $A_b := t_b\cdot h_b$

$A_{so} := n_{so}\cdot t_{so}\cdot h_{so}$ $A_{su} := n_{su}\cdot t_{su}\cdot h_{su}$

$E := 210GPa$

$A_{int} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.129\cdot m^2$ tab. 3.1

$A_c := A_{int} + A_d = 0.055\cdot m^2$ ref

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 345MPa & \text{if } 40mm < t \le 80mm \end{cases}$

Changed center of gravity, z_{tp}. From top to centre of gravity.

$z_{tp} := \frac{\begin{pmatrix} A_d\cdot\frac{t_d}{2} + A_{so}\left(t_d + \frac{h_{so}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ + A_b\left(t_d + h_s + h_{wy}\right) + A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right] \end{pmatrix}}{A_{tot}} = 516.076\cdot mm$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so}\cdot\frac{t_{so}\cdot h_{so}^3}{12} + n_{so}\cdot t_{so}\cdot h_{so}\left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 3.867\times 10^{-3}\cdot m^4$

$I_{y.st.su.1} := 6\cdot 1.506\cdot 10^{-6}\cdot m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2n_{su}\cdot h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 2.022\times 10^{-3}\cdot m^4$
$+ 2n_{su}\cdot h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$
$+ 2n_{su}\cdot h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2\cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w\cdot h_w\left[h_b - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 7.002\times 10^{-3}\cdot m^4$

Deck:

$I_{y.d} := \frac{h_d\cdot t_d^3}{12} + t_d\cdot h_d\left(\frac{t_d}{2} - z_{tp}\right)^2 = 9.142\times 10^{-3}\cdot m^4$

Bottom flange:

$I_{y.b} := \frac{h_b\cdot t_b^3}{12} + t_b\cdot h_b\left(h_b - z_{tp} - \frac{t_b}{2}\right)^2 = 7.546\times 10^{-3}\cdot m^4$

Side plate:

$I_{y.s} := 2\cdot\frac{t_s\cdot h_s^3}{12} + 2t_s\cdot h_s\left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 1.105\times 10^{-3}\cdot m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.031\cdot m^4$

$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \ge -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4\cdot\varepsilon(t)\cdot\sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055\cdot\max(3+\psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\cdot mm$

$z_2 := t_d + h_s = 720\cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.403$

$\text{Condition} := \begin{cases} 62\cdot\varepsilon\left(t_s\right)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \le -1 \\ \frac{42\cdot\varepsilon\left(t_s\right)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$

$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$k_{\sigma.4.1}(\psi_s) = 11.933$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.741$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 203.924\cdot mm$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 506.076\cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 506.076\cdot mm$

$h_{s.eff.c1} := 0.4\cdot h_{s.eff.c} = 202.43\cdot mm$

$h_{s.eff.c2} := 0.6\cdot h_{s.eff.c} = 303.646\cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 710\cdot mm$

$A_{s.eff.c} := 2\cdot h_{s.eff.c}\cdot t_s = 1.215\times 10^{4}\cdot mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right)\cdot t_s = 1.704\times 10^{4}\cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s)\cdot\frac{E\cdot\pi^2\cdot t_s^2}{12\left(1-\nu^2\right)\cdot h_s^2} = 646.957\cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$t_{sw} := t_d = 10\,mm$

$h_{sw} := t_d + b_{sw} = 150\,mm$

$\psi_{sw} := \frac{\sigma_{sw.d}(z_2)}{\sigma_{sw.d}(z_1)} = 0.723$

$k_{\sigma.4.2}(\psi_{sw}) = 0.544$ Higher stress close to support

$Class_{sw} := \begin{cases} \text{"Class4" if } \frac{h_{sw}}{t_{sw}} > 21 \cdot \varepsilon_{sw} \cdot \sqrt{k_{\sigma.4.2}(\psi_{sw})} \\ \text{"Lower than class 4" otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.sw} := \lambda_p\left(k_{\sigma.4.2}(\psi_{sw}), h_{sw}, t_{sw}\right) = 0.514$

$\rho_{d.sw} := \rho_{4.3}\left(\lambda_{p.sw}, \psi_d\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{sw.eff} := \rho_{d.sw} \cdot h_{sw} = 140\,mm$

$A_{sw.eff} := n_{sw} \cdot t_{sw} \cdot h_{sw.eff} = 2.016 \times 10^{-4}\,m^2$

Deck

$\frac{\lambda_{sw}}{t_d} = 35 \qquad 42 \cdot \frac{235MPa}{f_y(t_d)} = 34.172$

$Class4_{sw} := \begin{cases} \text{"Class4" if } \frac{\lambda_{sw}}{t_d} > 42 \cdot \frac{235MPa}{f_y(t_d)} \\ \text{"Lower than class 4" otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\,m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$\lambda_{sw} := \lambda_{sw} - \eta_{sw}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p\left(\sigma_{cr.d}, \varepsilon_{cm}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$z_{no.eff} := \rho_d \cdot z_{cn} = 321.461\,mm$

Areas:

$A_{sw} := n_{sw} \cdot t_{sw} \cdot t_d + n_{sw} \cdot t_{sw} \cdot \left(h_{sw.eff} + t_d\right) = 5.31 \times 10^{-4}\,m^2$

$A_{c.eff.loc} := n_{sw} \cdot t_{sw.eff} \cdot t_d + n_{sw} \cdot t_{sw} \cdot \left(h_{sw.eff} + t_d\right) = 5.053 \times 10^{-4}\,m^2$

More than three longitudinal stiffeners -> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^{-5}\,m^4$

$y_{tp.sl} := \frac{n_{sw} \cdot t_{sw} \cdot h_{sw} \cdot \left(t_d + \frac{h_{sw}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{sw} \cdot t_{sw} \cdot h_{sw} + b_d \cdot t_d} = 32.411\,mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \ldots + \left[\frac{n_{sw} \cdot t_{sw} \cdot h_{sw}^3}{12} + n_{sw} \cdot h_{sw} \cdot t_{sw} \cdot \left(t_d + \frac{h_{sw}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{-8}\,m^4$

$\gamma := \frac{\lambda_d}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{sw} \cdot h_{sw} \cdot t_{sw} = 0.02\,m^2$ EC1993-1-5 A.1 (2)

$k_{\sigma.p} := \begin{cases} \frac{2\left(1 + \alpha^2\right)^2 \cdot \gamma - 1}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt{\gamma} \\ \frac{4\left(1 - \frac{1}{\alpha}\sqrt{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 639.127$

$\delta := \frac{A_{sl}}{A_p} = 0.5$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_d^2} = 990.256\,MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{sw} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}\left(\lambda_p, 0\right) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, 0\right) \cdot A_{c.eff.loc} + z_{no.eff} \cdot t_d = 5.375 \times 10^{-4}\,m^2$

Bucking strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^{4}\,kN$

Column like behaviour

$\gamma_{tp.sl.c} := \frac{t_{sw} \cdot h_{sw} \cdot \left(t_d + \frac{h_{sw}}{2}\right) + n_{sw} \cdot t_d \cdot \frac{t_d}{2}}{t_{sw} \cdot h_{sw} + n_{sw} \cdot t_d} = 34.268\,mm$

$I_{sl.c} := \left[\frac{n_{sw} \cdot t_d^3}{12} + n_{sw} \cdot t_d \cdot \left(\frac{t_d}{2} - \gamma_{tp.sl.c}\right)^2\right] \ldots + \left[\frac{t_{sw} \cdot h_{sw}^3}{12} + h_{sw} \cdot t_{sw} \cdot \left(t_d + \frac{h_{sw}}{2} - \gamma_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{-7}\,m^4$

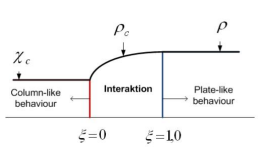
Buckling resistance stiffener

$A_{sw} := t_{sw} \cdot h_{sw} + x_{sw} \cdot t_d = 5.74 \times 10^{-3}\,mm^2$

$A_{sl.eff} := t_{sw} \cdot h_{no.eff} + z_{no.eff} \cdot t_d = 5.455 \times 10^{-3}\,mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot s^2} = 1.026 \times 10^{3}\,MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sw})}{\sigma_{cr.sl}}} = 0.573$

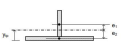
Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\,mm$

$e_2 := \max\left(\left(\gamma_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{sw}}{2} - \gamma_{tp.sl}\right)\right) = 47.589\,mm$



$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e_2}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot \left(\lambda_c - 0.2\right) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.pl}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaktion mellan de två olika beteendena(column/plate bucking):

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avsr} := n_{no} \cdot s_{no.eff} \cdot t_d \cdot \sigma_{neg}\left(t_d + \frac{t_d}{2}\right) + A_{sw} \cdot \sigma_{neg}\left(t_d + \frac{h_{no.eff}}{2}\right) = -1.622 \times 10^{4}\,kN$

$\sigma_{com.Ed} := \frac{F_{avsr}}{n_{no} \cdot s_{no.eff} \cdot t_d + A_{sw}} = -330.374\,MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sw})}{\gamma_M} = 281.032\,MPa$

$K_{A.2.1} := \begin{cases} \text{"OK" if } \frac{\rho_c \cdot f_y(t_{sw})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK" otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{no} \cdot s_{no.eff} \cdot t_{no.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{no.eff} + A_{d.eff} + A_w + A_{sw} + A_{fo} + A_b + A_{c.eff} = 0.126\,m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{no.eff} \cdot \left(t_d + \frac{h_{no.eff}}{2}\right) \ldots \\ &+ 2 \cdot \left[t_d \cdot h_{c.eff.c1} \cdot \left(t_d + \frac{h_{c.eff.c1}}{2}\right) \ldots \right. \\ &\left. + t_d \cdot h_{c.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{c.eff.c2}}{2}\right) + t_d \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \ldots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{sw} \cdot \left[t_d + h_s - \frac{2}{3} \cdot \left(h_{wy} - s_{no.ty}\right) - \frac{h_{no.y}}{2}\right] \ldots \\ &+ A_b \cdot \left(h_{tb} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 527.638\,mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_d \cdot h_{c.eff.c1}^3}{12} + t_d \cdot h_{c.eff.c1} \cdot \left(t_d + \frac{h_{c.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \ldots + \left[\frac{t_d \cdot h_{c.eff.c2}^3}{12} \ldots + t_d \cdot h_{c.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{c.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \ldots + A_{d.eff.neg} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right]\right] = 1.167 \times 10^{-3}\,m^4$

$A_{c.eff.loc} = 5.053 \times 10^{-4}\,m^2$

$I_{y.d.eff} := \frac{n_{sw} \cdot s_{no.eff} \cdot t_d^3}{12} + n_{sw} \cdot t_d \cdot s_{no.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 8.781 \times 10^{-7}\,m^4$

$I_{y.sl} := \frac{A_{c.eff.loc.mod} \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.883 \times 10^{3}\,mm^2$ If $I_{y.d.eff} = 0.031\,m^4$

Neg $A_{sl.eff.neg} := A_b$ Bottom flange in tension

Effective cross section with negative moment(when flange in tension)

The effective section is calculated according to A.2.1(5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is applied and the area above is used.

$A_{mod.loc} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{no.eff} \cdot t_d \text{ if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{no.eff} \cdot t_d \text{ otherwise} \end{cases} = 432.173\,cm^2$

A_{fo} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{sw} Stiffener class 4 --> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{sw} + A_b = 1.171 \times 10^{-3}\,cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d \text{ if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d \end{cases} = 7.916\,mm$

$z_{tp.eff} := \frac{\begin{aligned} &n_{no} \cdot s_{no.eff} \cdot t_{no} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{no}}{2}\right) \ldots \\ &+ s_{no.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{no} \cdot s_{no.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) \ldots \\ &+ t_d \cdot h_{c.eff.c1} \cdot \left(t_d + \frac{h_{c.eff.c1}}{2}\right) + t_d \cdot h_{c.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{c.eff.c2}}{2}\right) \ldots \\ &+ t_d \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots \\ &+ A_{sw} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{no.ty}\right) - \frac{h_{no.y}}{2}\right] + A_b \cdot \left(h_{tb} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 538.911\,mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{no} \cdot s_{no.eff} \cdot t_{d.eff}^3}{12} + t_{d.eff} \cdot n_{no} \cdot s_{no.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 9.164 \times 10^{-3}\,m^4$

$h_{web} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{no.fi} = 7.206\,mm$

Stiffeners over/under:

$I_{y.st.no.fin} := n_{no} \cdot \left[\frac{t_{no} \cdot s_{no.eff}^3}{12} + s_{no} \cdot t_{no} \cdot s_{no.eff} \cdot \left(t_d + \frac{s_{no.eff}}{2} - z_{tp.eff.neg}\right)^2\right] = 4.279 \times 10^{-3}\,m^4$

$I_{y.st.no.1.fin} = 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$I_{y.st.so.fin} := I_{y.st.no.1.fin} \ldots = 1.843 \times 10^{-3}\,m^4$
$+ 2 \cdot t_{sw} \cdot h_{sw} \cdot \left[t_d + h_s + \frac{1}{2} \cdot \left(h_{wy} - s_{no.ty}\right) - \frac{h_{no.y}}{2} - z_{tp.eff.neg}\right]^2 \ldots$
$+ 2 \cdot t_{sw} \cdot h_{sw} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{no.ty}\right) - \frac{h_{no.y}}{2} - z_{tp.eff.neg}\right]^2 \ldots$
$+ 2 \cdot t_{sw} \cdot h_{sw} \cdot \left[t_d + h_s + \left(h_{wy} - s_{no.ty}\right) - \frac{h_{no.y}}{2} - z_{tp.eff.neg}\right]^2$

Web:

$I_{y.w.1.fin} = 2 \cdot 0.000312\,m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[h_{tb} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 6.364 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y,b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_w - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 7.052 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ + \frac{t_s \cdot h_{s.c}^3}{12} + t_s \cdot h_{s.c} \cdot \left(t_d + h_s - \frac{h_{s.c}}{2} - z_{tp.eff.neg}\right)^2 \end{array} \right] = 1.231 \times 10^{-3}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 2.993 \times 10^{10}\ mm^4$

Load

Load capacity:

$q_M := 139 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_{Ve} when checking the moment capacity.

$Q_M := q_M \cdot b_d = 486.5 \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 1.97 \times 10^4\ kN \cdot m$

$q_V := 159 \frac{kN}{m^2}$ The load capacity is q_v when checking the shear capacity.

$Q_V := q_V \cdot b_d = 556.5 \frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 5.008 \times 10^3\ kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.056\ m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.972 \times 10^4\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.999$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon\left(t_w\right) \cdot t_w = 0.293\ m$

$\alpha_w := \min\left(\frac{h_{wy}}{b_w}, \frac{180}{\pi}\right) = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_b}{2} + s_{radi} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} \cdot \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right)}{2} \cdot h_{su} = 0.503\ m$

$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\ m$

$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\ m$

Moment of inirtia stiffeners z-direction

$I_{stw.z} := 2 b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\ m^4 \cdot m^4$ (two stiffeners)

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\ m^4$ reb area which is 15ct each side of the stiffeners:

Shear buckling coefficient, side plate

$\frac{h_{wy}}{2} + t_{radi} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\ m$

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{w.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{w.eff}} < 1 \end{cases} = 5.844$

$\lambda_{w1} := \frac{h_{w.eff}}{37.4 \cdot t_w \cdot \varepsilon\left(t_w\right) \cdot \sqrt{k_{\tau 1}}} = 0.804$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.493$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9 \cdot \left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\frac{I_{st.z.tot}}{\left(\min\left(t_w, t_{su}\right)^3 \cdot h_w\right)}}, \frac{2.1}{\min\left(t_w, t_{su}\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_w, t_{su}\right)^3 \cdot h_{web}} + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_w, t_{su}\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.204 \times 10^3$

α^2

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\frac{h_w}{t_w} = 1.2$ EC1993-1-3-5 ANM.2 5.1

$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_w, t_{su}\right)} \leq 31 \cdot \frac{\varepsilon\left(t_w\right)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 \cdot t_w \cdot \varepsilon\left(t_w\right) \cdot \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.804$ SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.032$ SS-EN 1993-1-5 5.3 Tabell 5.1, stiff end.

Kapacity of the web

$\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.2$ Safety class 3

$\gamma_{M1} = 1.00$ SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_a} = 2.506 \times 10^3\ kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 5.008 \times 10^3\ kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.999$ (contribution from two webs)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{tf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 533.911\ mm$

$z_{bf.eff.neg} := h_w - \frac{t_b}{2} - z_{tp.eff.neg} = 663.889\ mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \cdot \left\|z_{tf.eff.neg}\right\| \cdot f_y(t_d)}{\gamma_{M0}} \dots = 9.864 \times 10^3\ kN \cdot m$
$+ \frac{A_{bf.eff.neg} \cdot \left\|z_{bf.eff.neg}\right\| \cdot f_y(t_b)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.566\ m$

$c_{tf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.548\ m$

$V_{bf.Rd.cred} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{tf} \cdot \gamma_{M1} \cdot \gamma_a} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{tf} \cdot \gamma_{M1} \cdot \gamma_a} = 321.269\ kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq 5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cred} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} = 0\ kN \\ 0 & \text{otherwise} \end{cases}$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{Rd.Rek} := V_{bw.Rd} + V_{bf.Rd} = 2.506 \times 10^3\ kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.999$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.016\ m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{su} \cdot s_{su.eff} \cdot t_{d.eff} + n_{su} \cdot s_{st} \cdot t_{d.eff} + A_b = 432.579\ cm^2$

$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{su} \cdot s_{su.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \dots}{+ n_{su} \cdot s_{st} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_w - \frac{t_b}{2}\right)} = 447.457\ mm$
$A_{eff.f.tot}$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.036\ m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.287 \times 10^4\ kN \cdot m$

Shear capacity check when $q_V = 159 \frac{kN}{m^2}$

Results

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.999$

$M_{pl.Rd} := M_{Rd}$

Moment capacity check when $q_M = 139 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.999$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

The Reduced stress method

-Changed side plate height to 710 mm

Material

$f_{y0} := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ 335\,mm & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$k = 210\,GPa$

$\nu = 0.3$ $G = \frac{k}{2\,(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$t_b = 16mm$ $b_b = 1000mm$ $A_b = b_b t_b = 0.016m^2$

Top plate

$t_d = 18mm$ $b_d = 3500mm$ $A_d = b_d t_d = 0.035m^2$

Side plates

$t_s = 12mm$ $h_s = 710mm$ $A_s = h_s t_s = 8.52 \cdot 10^{-3} m^2$

Webs

$t_w = 12mm$ $h_w = 1340mm$ $A_w = h_w t_w = 0.016m^2$

$h_{w,n} = 1250mm$ $h_{w,p} = 482.8mm$

Top stiffeners

$t_{st} = 16mm$ $b_{st} = 140mm$ $A_{st} = h_{st} t_{st} = 2.24 \cdot 10^{-3} m^2$

$s_{st} = 350mm$ $n_{st} = 9$

Web stiffeners

$t_{wst} = 12mm$ $h_{wst} = 120mm$ $A_{wst} = h_{wst} t_{wst} = 1.44 \cdot 10^{-3} m^2$

$s_{wst} = 440mm$ $n_{wst} = 3$

Whole bridge

$L_{bridge} = 18m$ $y_b = \frac{h_s}{2} + h_{w,p} + t_b + t_d = 1.211m$ $z_b = 2m$

$I_{yy} = 0.030663m^4$ From grasshopper

$V_{yy} = 0.694881m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q = 110\,\frac{kN}{m^2}$

$Q = q\,z_b = 385\,\frac{kN}{m}$

$M_{Ed} = \frac{Q\,L_{bridge}^2}{8} = 1.559 \cdot 10^4\,kN\,m$

$V_{Ed} = \frac{Q\,L_{bridge}}{2} = 3.465 \cdot 10^3\,kN$

Top plate

$\sigma_{x,Ed,top} = \frac{M_{Ed}}{I_{yy}}\left(h_s - \frac{t_d}{2} - y_b\right) = 259.808\,MPa$ $\sigma_{x,Ed,typ} = 0$

$S_{yy} = \frac{t_d\,b_d}{n_w + 2}\left(h_s - \frac{t_d}{2} - y_b\right) = 1.626 \cdot 10^{-3} m^3$

$\tau_{Ed,typ} = \frac{V_{Ed}\,S_{yy}}{I_{yy}\,\min(t_w\,t_{st})} = 15.309\,MPa$

Side plates

$\sigma_{x,Ed,sup} = \frac{M_{Ed}}{I_{yy}}\left(h_s - t_d - y_b\right) = 257.266\,MPa$

$\sigma_{z,Ed,sup} = \frac{qb}{n_s} = 16.042\,MPa$

$S_{yy} = \left(h_s - t_d - y_b\right) t_s \left(\frac{h_s - t_d - y_b}{2} + \frac{t_d\,b_d}{2}\left(h_s - \frac{t_d}{2} - y_b\right)\right) = 0.015m^3$
$+ \frac{h_{st}}{2} t_{st} t_{st}\left(h_s - t_d - \frac{h_{st}}{2} - y_b\right)$

$S_{yy,s} = \frac{\left(h_w \frac{t_w\,h_w}{2} + 2\,\frac{t_s\,h_s}{2}\right)}{n_w + 2}\left(h_s - t_d - \frac{h_w}{4} - y_b\right) = 2.129 \cdot 10^{-3} m^3$
$+ \frac{t_d\,b_d}{n_w + 2}\left(h_s - \frac{t_d}{2} - y_b\right)$

Tim nästa stiffenern:

$\tau_{Ed,typ} = \frac{V_{Ed}\,S_{yy}}{I_{yy}\,t_w} = 15.037\,MPa$

$\sigma_{x,Ed,typ} = \frac{M_{Ed}}{I_{yy}}\left(h_s - t_d - y_b\right) = 257.266\,MPa$

Governing slenderness from FE-software

$\sigma_{V,1,Ed} = 3.48\,\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_{cr} = \sigma_{V,1,Ed}\,c = 349.8\,MPa$

$\alpha_{ult} = \frac{\lambda_1(\psi)}{\sigma_{cr}} = 1.015$

$\sigma_{cr,1,Ed} = 274.42\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$c_{cr} = \frac{\sigma_{cr,1,Ed}}{c} = 2.495$

$\lambda_p = \sqrt{\frac{f_{y0}}{\sigma_{cr}}} = 0.638$ Slenderness for all parts

Top plate check

$\psi_{typ} = 1$

$\rho_{p,typ} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\,(3 + \psi_{typ})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E,typ} = \frac{\pi^2\,k_1}{12\left(1-\nu^2\right)\left(\frac{b_d}{t_d}\right)^2} = 1.549\,MPa$

$I_{yy,typ} = \frac{b_d\,t_d^3}{12\left(1-\nu^2\right)} = 3.205 \cdot 10^{-7} m^4$

$z_{g,d,typ} = \frac{b_d\,\frac{t_d}{2} + n_w\,h_w\left(t_d + \frac{h_w}{2}\right)}{b_d + n_w\,h_w} = 32.411\,mm$

$I_{d,typ} = \frac{b_d\,t_d^3}{12} + b_d\left(z_{g,d,typ} - \frac{t_d}{2}\right)^2 \cdot \ldots = 1.052 \cdot 10^{-5} mm^4$
$+ n_w\left[\frac{t_w\,h_w^3}{12} + h_w\left(t_d + \frac{h_w}{2} - z_{g,d,typ}\right)^2\right]$

$\gamma_{typ} = \frac{I_{d,typ}}{I_{yy,typ}} = 328.143$

$k_{\sigma,typ} = \frac{n_w\,h_w}{b_d} = 0.576$

$\alpha_{typ} = \min\left(\frac{2a}{h_w}, 0.5\right) = 0.571$ $\sqrt{\gamma_{typ}} = 4.256$

$k_{\sigma,typ} = \begin{cases} \left[2\left(1+\alpha_{typ}\right)^2 - \alpha_{typ}^2 - 1\right] & \text{if } \alpha_{typ} < \sqrt{\gamma_{typ}} \\ \alpha_{typ}^2\left(\alpha_{typ}-1\right)\left(1+\psi_{typ}\right) & \\ 4\left(1+\sqrt{\gamma_{typ}}\right) & \text{if } \alpha_{typ} \ge \sqrt{\gamma_{typ}} \\ \left(\alpha_{typ}-1\right)\left(1+\psi_{typ}\right) & \end{cases} = 639.127$

$\sigma_{cr,typ} = k_{\sigma,typ}\,\sigma_{E,typ} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1,ad,typ} = \frac{3 - \psi_{typ}}{5 - \psi_{typ}}\,b_{ad} = 0.175m$

$b_{2,sup,typ} = \frac{2}{5 - \psi_{typ}}\,b_{ad} = 0.175m$ Parts of the top plate that contributes to the stiffener

$b_{d,1,typ} = b_{1,ad,typ} + b_{2,sup,typ} = 0.35m$

$z_{g,d,1,typ} = \frac{t_d\,b_{d,1,typ}\,\frac{t_d}{2} + A_{wst}\left(t_d + \frac{h_{wst}}{2}\right)}{t_d\,b_{d,1,typ} + A_{wst}} = 0.034m$ combined center of gravity for stiffener and contributing top plate.

$I_{d,1,typ} = \frac{b_{d,1,typ}\,t_d^3}{12} + b_{d,1,typ}\,t_d\left(z_{g,d,1,typ} - \frac{t_d}{2}\right)^2 + \frac{t_{wst}\,h_{wst}^3}{12} + A_{wst}\left(t_d + \frac{h_{wst}}{2} - z_{g,d,1,typ}\right)^2$

Combined moment of inertia

$\sigma_{cr,d,typ} = \frac{\pi^2\,k_{1,d,typ}}{b_{d,1,typ}\,t_d^2} \cdot 1.026 \cdot 10^3\,MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,w} = \sigma_{cr,d}$

$\sigma_{cr,c,typ} = \sigma_{cr,d,typ}$

Reduction factor for column-like behaviour

$\gamma_{typ} = \sqrt{\frac{I_{d,1,typ}}{A_{d,1,typ}}} = 0.045m$

$e_{typ} = \max\left(e_{typ,d,1,typ} - z_{g,1,typ} + \frac{h_{wst}}{2} - z_{typ,d,1,typ}\right) = 0.046m$

$\alpha_{e,typ} = 0.49 + \frac{0.09}{\frac{\gamma_{typ}}{e_{typ}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{typ} = 0.5\left[1 + \alpha_{e,typ}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.831$ [6.3.1.2]

$\chi_{typ} = \min\left(\frac{1}{\phi_{typ} + \sqrt{\phi_{typ}^2 - \lambda_p^2}}, 1\right) = 0.733$

Interpolation

$t_{\psi,typ} = \min\left(\max\left(\frac{\sigma_{c,typ,typ}}{\sigma_{cr,c,typ}} - 1.0, 0\right), 1\right) = 0$

$\rho_{c,typ} = \begin{cases} \left[\left(\rho_{p,typ} - \chi_{typ}\right)\left(t_{\psi,typ}\left(2 - t_{\psi,typ}\right) - \chi_{typ}\right)^2 \text{ if } \frac{\left[\left(\rho_{p,typ} - \chi_{typ}\right)\left(t_{\psi,typ}\left(2 - t_{\psi,typ}\right) - \chi_{typ}\right)^2\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{p,typ} - \chi_{typ}\right)\left(t_{\psi,typ}\left(2 - t_{\psi,typ}\right) - \chi_{typ}\right)^2\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.733$

Shear

$\eta_1 = 1.2$ 7.1 (2)

$\chi_w = \begin{cases} \eta_1 & \text{if } \lambda_p < \frac{0.83}{\eta_1} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_1} \le \lambda_p < 1.08 \\ \frac{1.37}{\left(0.7 + \lambda_p\right)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$

$\frac{\tau_{Ed,typ}\,\alpha_{ult}}{\gamma_{M1}} = 0.744$

Side plates check

x-direction

$\sigma_{max} = f_y(\psi)$

$\sigma_{E1} = \left|\frac{-\sigma_{max}}{f_y(\psi)}\right| + \sigma_{max}\Big| - 1$

$\psi_{sp,x} = \frac{\sigma\left(h_s - t_d - h_1\right)}{\sigma\left(h_s - t_d\right)} = -0.403$

$\sigma_{x,sup} = \frac{M_{Ed}}{I_{yy}}\left(h_s - t_d - y_b\right) = 257.266\,MPa$

$\sigma_{x,bot} = \frac{M_{Ed}}{I_{yy}}\left(h_s - t_d - h_s - y_b\right) = -103.778\,MPa$

$\psi_{sp1,x} = \max\left(3 \cdot \psi_{sp,x}, 0\right) = 2.597$ $\psi_{sp1} = \frac{\sigma_{x,bot}}{\sigma_{x,sup}} = -0.403$

$\rho_{p,sp,x} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055\,(\psi_{sp1,x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,sp,x} = \begin{cases} 1 & \text{if } \psi_{sp,x} \ge 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp,x}\right)} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{sp,x} = 0 \\ 7.81 - 6.29\,\psi_{sp,x} + 9.78\,\psi_{sp,x}^2 & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{sp,x} = -1 \\ 5.98\left(1 - \psi_{sp,x}\right)^2 & \text{if } -1 > \psi_{sp,x} > -3 \\ 95.68 & \text{if } \psi_{sp,x} < -3 \end{cases} = 11.939$

$\sigma_{cr,sp,x} = \sigma_{E,sp,x}\,k_{\sigma,sp,x} = 647.291\,MPa$

Column-like behaviour

$\sigma_{E,c,sp,x} = \frac{\pi^2\,k_1\,t_s^2}{12\left(1-\nu^2\right)\,h_s^2} = 6.833\,MPa$

$\alpha_{sp} = 0.21$ $\phi_{sp} = 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.749$

$\chi_{sp} = \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1\right) = 0.875$

Interpolation

$t_{\psi,sp,x} = \min\left(\max\left(\frac{\sigma_{c,sp,sp,x}}{\sigma_{cr,c,sp,x}} - 1.0, 0\right), 1\right) = 1$

$\rho_{c,sp,x} = \begin{cases} \left[\left(\rho_{p,sp} - \chi_{sp}\right)\left(t_{\psi,sp,x}\left(2 - t_{\psi,sp,x}\right) - \chi_{sp}\right)^2 \text{ if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\left(t_{\psi,sp,x}\left(2 - t_{\psi,sp,x}\right) - \chi_{sp}\right)^2\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{p,sp} - \chi_{sp}\right)\left(t_{\psi,sp,x}\left(2 - t_{\psi,sp,x}\right) - \chi_{sp}\right)^2\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

z-direction

$\psi_{sp,z} = 1$

$\alpha_{sp,sp} = 0.34$ $\chi_{sp,sp} = 0.7$

$\phi_{sp,sp} = 0.5\left[1 + \alpha_{sp,sp}\left(\lambda_p - \chi_{sp,sp}\right) + \lambda_p^2\right] = 0.808$

$\rho_{p,sp,sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{sp,sp} + \sqrt{\phi_{sp,sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,sp,sp,z} = 4$

$\sigma_{E,sp,z} = \frac{\pi^2\,k_{\sigma,z}\,t_s^2}{12\left(1-\nu^2\right)\,a_{st}^2} = 6.833\,MPa$

$\sigma_{cr,sp,sp,z} = \lambda_{sp,sp,sp,z}\,\sigma_{E,sp,z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{E,c,sp,sp,z} = \frac{\pi^2\,k_1\,t_s^2}{12\left(1-\nu^2\right)\,h_s^2} = 54.218\,MPa$

Interpolation

$t_{\psi,sp,z} = \min\left(\max\left(\frac{\sigma_{c,sp,sp,z}}{\sigma_{cr,c,sp,sp,z}} - 1.0, 0\right), 1\right) = 0$

$\rho_{c,sp,z} = \begin{cases} \left[\left(\rho_{p,sp,sp} - \chi_{sp,sp}\right)\left(t_{\psi,sp,z}\left(2 - t_{\psi,sp,z}\right) - \chi_{sp,sp}\right)^2 \text{ if } \frac{\left[\left(\rho_{p,sp,sp} - \chi_{sp,sp}\right)\left(t_{\psi,sp,z}\left(2 - t_{\psi,sp,z}\right) - \chi_{sp,sp}\right)^2\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{p,sp,sp} - \chi_{sp,sp}\right)\left(t_{\psi,sp,z}\left(2 - t_{\psi,sp,z}\right) - \chi_{sp,sp}\right)^2\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.875$

Top plate stiffeners - check

x-direction

$$\rho_{p,x,ips} := \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{ips,x} := \frac{\sigma(h_w - t_f - h_{nc})}{\sigma(h_w - t_f)} = 0.723$$

$$\sigma_{E,ips,x} := \frac{\pi^2 \cdot E \cdot t_{ips}^2}{12 \cdot (1 - \nu^2) \cdot b_{ips}^2} = 2.479 \times 10^3 \cdot MPa$$

$$k_{\sigma,ips,x} := \begin{cases} 1 & \text{if } \psi_{ips,x} \geq 1 \\ \frac{8.2}{1.05 + \psi_{ips,x}} & \text{if } 1 > \psi_{ips,x} > 0 \\ 7.81 & \text{if } \psi_{ips,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{ips,x} + 9.78 \cdot \psi_{ips,x}^2 & \text{if } 0 > \psi_{ips,x} > -1 \\ 23.9 & \text{if } \psi_{ips,x} = -1 \\ 5.98 \cdot (1 - \psi_{ips,x})^2 & \text{if } -1 > \psi_{ips,x} > -3 \\ 95.68 & \text{if } \psi_{ips,x} < -3 \end{cases} = 4.852$$

$$\sigma_{cr,ip,ips,x} := k_{\sigma,ips,x} \cdot \sigma_{E,ips,x} = 1.203 \times 10^4 \cdot MPa$$

Column-like behaviour

$$\sigma_{c.c.c.ips,x} := \frac{\pi^2 \cdot E \cdot t_{ips}^2}{12 \cdot (1 - \nu^2) \cdot s_p^2} = 12.147 \cdot MPa$$

$$\lambda_{ips} := \lambda_p$$

Interpolation

$$\xi_{ips,x} := \min\left(\max\left(\frac{\sigma_{cr,ip,ips,x}}{\sigma_{c.c.c.ips,x}} - 1.0\right), 1\right) = 1$$

$$\rho_{c,ips,x} := \begin{cases} \left[1 - \left(\rho_{ips,ips} - \chi_{ips}\right) \cdot \xi_{ips,x} \cdot \left(2 - \xi_{ips,x}\right) - \chi_{ips}\right] & \text{if } \frac{\left[\left(\rho_{ips,ips} - \chi_{ips}\right) \cdot \xi_{ips,x} \cdot \left(2 - \xi_{ips,x}\right) + \chi_{ips}\right] \cdot \sigma_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{ips,ips} - \chi_{ips}\right) \cdot \xi_{ips,x} \cdot \left(2 - \xi_{ips,x}\right) + \chi_{ips}\right] \cdot \sigma_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.ips}}{\frac{\rho_{c,ips,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.ips}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 1$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right) = 0.571$$
$$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c,sp,x} \cdot f_y}{\gamma_{M1}}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.sp}}{\frac{\chi_w \cdot f_y}{\gamma_{M1}}}\right)^2 = 0.529$$

EWM - Changed side plate height

This chapter covers the EWM when changing the side plate height to 790 mm

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s,so} := n_{so} + 1 = 10$

Deck: $h_d := 3500mm$ $t_d := 10mm$

$h_{d,0.5} := 1750mm$

$s_{so} := \frac{h_d}{n_{s,so}} = 350 \cdot mm$

Side plate: $h_s := 790mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{so0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $h_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su,y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 1.291 \times 10^3 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts,1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 1.265 \times 10^3 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d \cdot h_{\text{uarameter}}$ $A_s := 2 t_s h_s$ $A_w := 2 t_w h_w$ $A_b := t_b h_b$

$A_{so} := n_{so} t_{so} h_{so}$ $A_{su} := n_{su} t_{su} h_{su}$

$E := 210GPa$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.133 \cdot m^2$ tab. 3.1

$A_c := A_{tot}$

$A_c = A_{tot} + A_d = 0.055 \cdot m^2$ ok

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 345MPa & \text{if } 40mm < t \le 80mm \end{cases}$

Changed center of gravity, z_{cg}

$$z_{cg} := \frac{A_d \frac{t_d}{2} + A_{so}\left(t_d + \frac{h_{so}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_b\left(t_d + h_s + h_{wy}\right) + A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{so0}\right) - \frac{h_{su,y}}{2}\right]}{A_{tot}} = 554.361 \cdot mm$$

Moment of inertia I_y

$s_{so0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{so0} = 7.206 \cdot mm$

Stiffeners overlunder:

$I_{y,st.so} := n_{so}\left[\frac{t_{so} h_{so}^3}{12} + h_{so} t_{so}\left(t_d + \frac{h_{so}}{2} - z_{cg}\right)^2\right] = 4.569 \times 10^{-3} \cdot m^4$

$I_{y,st.su.1} := 6 \cdot 1.506 \cdot 10^{-5} \cdot m^4$ from FEM

$$I_{y,st.su} := I_{y,st.su.1} + 2 t_{su} h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{so0y}\right) - \frac{h_{su,y}}{2} - z_{cg}\right]^2 + 2 t_{su} h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{so0y}\right) - \frac{h_{su,y}}{2} - z_{cg}\right]^2 + 2 t_{su} h_{su}\left[t_d + h_s + \left(h_{wy} - s_{so0y}\right) - \frac{h_{su,y}}{2} - z_{cg}\right]^2 = 2.372 \times 10^{-3} \cdot m^4$$

Web:

$I_{y,w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y,w} := I_{y,w.1} + 2 t_w h_w\left[h_{ts} - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{cg}\right]^2 = 8.253 \times 10^{-3} \cdot m^4$

Deck:

$I_{y,d} := \frac{h_d t_d^3}{12} + t_d h_d\left(\frac{t_d}{2} - z_{cg}\right)^2 = 0.011 \cdot m^4$

Bottom flange:

$I_{y,b} := \frac{h_b t_b^3}{12} + h_b t_b\left(h_{ts} - z_{cg} - \frac{t_b}{2}\right)^2 = 8.49 \times 10^{-3} \cdot m^4$

Side plate:

$I_{y,s} := 2\left[\frac{t_s h_s^3}{12} + 2 t_s h_s\left(t_d + \frac{h_s}{2} - z_{cg}\right)^2\right] = 1.409 \times 10^{-3} \cdot m^4$

Total:

$I_{y,tot} := I_{y,st.so} + I_{y,st.su} + I_{y,w} + I_{y,d} + I_{y,b} + I_{y,s} = 0.036 \cdot m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{cg}} z - \sigma_{max}$

$\sigma_{1,neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2,neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{cg}} z + \sigma_{max}$

$\sigma_{1,pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2,pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2,neg}(z_1, z_2)}{\sigma_{1,neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2,pos}(z_1, z_2)}{\sigma_{1,pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \ge -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(b_{pl}, t, \epsilon) := \frac{\frac{b}{t}}{28.4 \cdot \epsilon(\psi) \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 800 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.451$

$Condition := \begin{cases} 62 \cdot \epsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \le -1 \\ 42 \cdot \epsilon(t_s) & \text{otherwise} \\ 0.67 + 0.33 \cdot \psi_s \end{cases}$

$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$k_{\sigma.4.1}(\psi_s) = 12.64$

$s_{c,so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.801$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 245.639 \cdot mm$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 544.361 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 544.361 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 217.744 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 326.617 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 790 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 1.306 \times 10^4 \cdot mm^2$

$A_{s.eff} := 2\left(h_{s.eff.c} + h_{s.t}\right) \cdot t_s = 1.896 \times 10^4 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \frac{E \cdot \pi^2 \cdot s_{so}^2}{12\left(1 - \nu^2\right) h_s^2} = 553.531 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$t_{sw} := t_d = 10\,mm$

$h_{sw} := t_d + h_{ss} = 150\,mm$

$\psi_{ss} := \frac{\sigma_{sup}(z_2)}{\sigma_{sup}(z_1)} = 0.743$

$k_{\sigma.4.2}(\psi_{ss}) = 0.534$ Higher stress close to support

$Class4_{ss} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{ss}}{t_{ss}} > 21 \cdot \varepsilon \cdot \sqrt{k_{\sigma.4.2}(\psi_{ss})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.ss} := \lambda_p(k_{\sigma.4.2}(\psi_{ss}), b_{ss}, t_{ss}) = 0.518$

$\rho_{d.ss} := \rho_{4.2}(\lambda_{p.ss}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{ss.eff} := \rho_{d.ss} \cdot b_{ss} = 140\,mm$

$A_{ss.eff} := n_{ss} \cdot b_{ss.eff} \cdot t_{ss} = 2.016 \times 10^{-4}\,m^2$

Deck

$\frac{b_{ss}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{ss} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{ss}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\,m \qquad \psi_d := \psi_{sup}(z_d, z_d) = 1$

$k_{sw} := k_{ss} = 4$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.ss}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$s_{ss.eff} := \rho_d \cdot s_{c.ss} = 321.461\,mm$

Areas:

$A_{ss} := n_{ss} \cdot s_{ss} \cdot t_d + n_{ss} \cdot t_{ss} \cdot (h_{ss.eff} + t_d) = 5.31 \times 10^{-4}\,m^2$

$A_{c.eff.loc} := n_{ss} \cdot s_{ss.eff} \cdot t_d + n_{ss} \cdot t_{ss} \cdot (h_{ss.eff} + t_d) = 5.053 \times 10^{-4}\,m^2$

More than three longitudinal stiffeners --> $k_{\sigma.sp}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5}\,m^4$

$y_{sp.sl} := \frac{n_{ss} \cdot t_{ss} \cdot h_{ss} \cdot \left(t_d + \frac{h_{ss}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{ss} \cdot t_{ss} \cdot h_{ss} + b_d \cdot t_d} = 32.411\,mm$

$I_d := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl}\right)^2\right] \dots = 1.052 \times 10^{-8}\,m^4$
$+ \left[\frac{n_{ss} \cdot t_{ss} \cdot h_{ss}^3}{12} + n_{ss} \cdot h_{ss} \cdot t_{ss} \cdot \left(t_d + \frac{h_{ss}}{2} - y_{sp.sl}\right)^2\right]$

$\gamma := \frac{I_d}{I_p} = 328.143 \qquad \alpha_{sp} := \frac{a}{b} = 0.571$

$k_{\sigma.sp} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt{\gamma} \\ \frac{4 \cdot \left[1 + \sqrt{\gamma}\right]}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 639.127$ 2)

$A_{sw.loc} = 0.0035\,m^2$

Check buckling stress, orthotropic plate:

$\sigma_{cr.pl} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot i_d^2}{12 \cdot (1 - \nu^2) \cdot b_d^2} = 990.256\,MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{pl} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{ss.eff} \cdot t_d = 5.375 \times 10^{-4}\,m^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^{4}\,kN$

Column like behaviour

$y_{sp.sl.c} := \frac{n_{ss} \cdot h_{ss} \cdot \left(t_d + \frac{h_{ss}}{2}\right) + s_{ss} \cdot t_d \cdot \frac{t_d}{2}}{n_{ss} \cdot h_{ss} + s_{ss} \cdot t_d} = 34.268\,mm$

$I_{d.c} := \left[\frac{s_{ss} \cdot t_d^3}{12} + s_{ss} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl.c}\right)^2\right] \dots = 1.137 \times 10^{-7}\,m^4$
$- \left[\frac{t_{ss} \cdot h_{ss}^3}{12} + h_{ss} \cdot t_{ss} \cdot \left(t_d + \frac{h_{ss}}{2} - y_{sp.sl.c}\right)^2\right]$

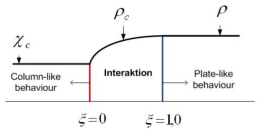
Buckling resistance stiffener

$A_{sl} := t_{ss} \cdot h_{ss} + s_{ss} \cdot t_d = 5.74 \times 10^{-4}\,m^2$

$A_{sl.eff} := t_{ss} \cdot h_{ss.eff} + s_{ss.eff} \cdot t_d = 5.455 \times 10^{-4}\,m^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot i_{sl.c}}{A_{sl} \cdot s^2} = 1.026 \times 10^{3}\,MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sw})}{\sigma_{cr.sl}}} = 0.573$

calculation of reduction factor EC1993-1-5 4.5.3

$\rho_{ss} = 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\,mm$

$z_{sl} := \max\left(\left[y_{sp.sl} - \frac{t_d}{2}\right], \left(t_d + \frac{h_{ss}}{2} - y_{sp.sl}\right)\right) = 47.589\,mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{z_{sl}}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot \left(\lambda_c - 0.2\right) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{c.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaktion mellan de två olika beteendena(column/plate bucking):

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{axial} := n_{ss} \cdot s_{ss.eff} \cdot t_d \cdot \sigma_{sup}\left(\frac{t_d}{2}\right) + A_{ss} \cdot \sigma_{sup}\left(t_d + \frac{h_{ss.eff}}{2}\right) = -1.63 \times 10^{4}\,kN$

$\sigma_{com.Ed} := \frac{F_{axial}}{n_{ss} \cdot s_{ss.eff} \cdot t_d + A_{ss}} = -332.075\,MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sw})}{\gamma_M} = 281.032\,MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{ss} \cdot s_{ss} \cdot t_{ss.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{ss.eff} + A_{d.eff} + A_w + A_{ss} + A_b + A_{s.eff} = 0.128\,m^2$

$z_{sp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{ss.eff} \cdot \left(t_d + \frac{h_{ss.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ &\left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{ss} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{ssfy}\right) - \frac{h_{ss.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{tb} - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 566.604\,mm$

$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{sp.eff.neg}\right)^2 \dots \\ &+ \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c2}^3}{12} \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{sp.eff.neg}\right)^2 \dots \\ &\left. + \left[\frac{t_s \cdot h_s^3}{12} + t_s \cdot h_s \cdot \left(t_d + h_s - \frac{h_s}{2} - z_{sp.eff.neg}\right)^2\right]\right] \dots \end{aligned}\right] = 1.481 \times 10^{-3}\,m^4$

Effective cross section with negative moment (lower flange in tension)

Area

$A_{tot.eff.neg} := A_{d.eff} + n_{ss} \cdot s_{ss} \cdot t_d + n_{ss} \cdot s_{ss} \cdot t_{ss.eff} = 0.032\,m^2$

$I_{s.d.eff} := \frac{s_{ss} \cdot t_d^3}{12} + n_{ss} \cdot s_{ss} \cdot t_d \cdot \left(\frac{t_d}{2} - z_{sp.eff.neg}\right)^2 = 0.01\,m^4$

$A_{c.eff.loc} = 5.053 \times 10^{-4}\,mm^2$

$I_{A_{c.eff.loc.mod}} := \frac{\rho_c \cdot f_y(t_{sl}) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.858 \times 10^{3}\,mm^2 \cdot s_{s.eff} + I_{y.d.eff} = 0.037\,m^4$

$A_{sl.eff.neg} := A_b$ Bottom flange in tension

Negative moment

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{web.c} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{ss.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{ss.eff} \cdot t_d & \text{otherwise} \end{cases} = 432.173\,cm^2$

A_{sl} Lower stiffener in tension -> no reduction

A_b Tension -> No reduction

A_s No reduction if the side plate is in class 4.

A_{ss} Stiffener class 4 -> no reduction

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_w + A_{ss} + A_b = 1.19 \times 10^{-3}\,cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.916\,mm$

$z_{sp.eff} := \frac{\begin{aligned} &n_{ss} \cdot s_{ss.eff} \cdot t_{ss} \cdot \frac{t_{d.eff}}{2} \cdot \left(t_d + \frac{h_{ss}}{2}\right) \dots \\ &+ s_{ss.eff} \cdot t_d \cdot \frac{t_d}{2} + s_{ss} \cdot s_{ss.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) \dots \\ &+ t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ &+ A_{ss} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{ssfy}\right) - \frac{h_{ss.y}}{2}\right] + A_b \cdot \left(h_{tb} - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 574.539\,mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{ss} \cdot s_{ss.eff} \cdot t_{d.eff}^3}{12} + t_d \cdot n_{s.ss} \cdot s_{ss.eff} \cdot \left(\frac{t_d}{2} - z_{sp.eff.neg}\right)^2 = 0.01\,m^4$

$s_{ss.eff} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{ss0} = 7.206\,mm$

Stiffeners over/under:

$I_{y.st.ss.fin} := n_{ss} \cdot \frac{s_{ss} \cdot h_{ss.eff}^3}{12} + n_{ss} \cdot t_{ss} \cdot h_{ss.eff} \cdot \left(t_d + \frac{h_{ss.eff}}{2} - z_{sp.eff.neg}\right)^2 = 4.963 \times 10^{-3}\,m^4$

$I_{y.st.ss.1.fin} = 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$I_{y.st.ss.fin} := I_{y.st.ss.1.fin} \dots = 2.199 \times 10^{-3}\,m^4$
$+ 2 \cdot t_{ss} \cdot h_{ss} \cdot \left[t_d + h_s + \frac{1}{3} \cdot \left(h_{wy} - s_{ssfy}\right) - \frac{h_{ss.y}}{2} - z_{sp.eff.neg}\right]^2 \dots$
$+ 2 \cdot t_{ss} \cdot h_{ss} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{ssfy}\right) - \frac{h_{ss.y}}{2} - z_{sp.eff.neg}\right]^2 \dots$
$+ 2 \cdot t_{ss} \cdot h_{ss} \cdot \left[t_d + h_s + \left(h_{wy} - s_{ssfy}\right) - \frac{h_{ss.y}}{2} - z_{sp.eff.neg}\right]^2$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w h_w \left[h_n - \frac{h_b}{2} - \frac{1}{2}(h_{wy}) - z_{tp.eff.neg} \right]^2 = 7.634 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b h_b^3}{12} + b_b h_b \left(h_n - z_{tp.eff.neg} - \frac{h_b}{2} \right)^2 = 8.026 \times 10^{-3} m^4$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s h_{s.eff.c.1}^3}{12} + t_s h_{s.eff.c.1} \left(t_d + \frac{h_{s.eff.c.1}}{2} - z_{tp.eff.neg} \right)^2 \dots \\ + \frac{t_s h_{s.eff.c.2}^3}{12} + t_s h_{s.eff.c.2} \left(t_d + h_{s.c} - \frac{h_{s.eff.c.2}}{2} - z_{tp.eff.neg} \right)^2 \dots \\ + \frac{t_s h_{s.t}^3}{12} + t_s h_{s.t} \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg} \right)^2 \end{array} \right] = 1.531 \times 10^{-3} m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 3.478 \times 10^{-3} m^4$

Load

Load capacity:

$q_{_M} := 151 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_{dp} when checking the moment capacity.

$Q_{_M} := q_{_M} \cdot b_d = 528.5 \frac{kN}{m}$

$M_{Ed} := \frac{Q_{_M} \cdot l_{bridge}^2}{8} = 2.14 \times 10^4 kN \cdot m$

$q_{_V} := 154 \frac{kN}{m^2}$ The load capacity is q_v when checking the shear capacity.

$Q_{_V} := q_{_V} \cdot b_d = 539 \frac{kN}{m}$

$V_{Ed} := \frac{Q_{_V} \cdot l_{bridge}}{2} = 4.851 \times 10^3 kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.061 m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 2.149 \times 10^4 kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.996$

Shear capacity

$\eta = 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeners

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$a_{st.z.3} := a_{st.z.2} + s_{st} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324m$

Moment of inertia about z-axis in z-direction Angle of web

$I_{st.z} := 2 t_{st} h_{st} \left[\left(a_{st.z.1} \right)^2 + \left(a_{st.z.2} \right)^2 + \left(a_{st.z.3} \right)^2 \right] = 8.18 \times 10^{-3} m^4$ (two stiffeners)

Distance to stiffeners from z-axis

Distance from z-axis to middle of the web around which the stiffeners are placed

$a_{w.z.1} := \frac{h_w}{2} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{flat}}{2} = 0.583m$

$a_{w.z.2} := a_{w.z.1} + s_{st} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{flat}}{2} = 0.525m$

$a_{w.z.1} := a_{st.z.1} + s_{st} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935m$

$a_{w.z.2} := a_{st.z.2} + s_{st} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346m$

Moment of inertia of 15st of the web z-direction

$I_{stw.z} := 2 h_{web} \cdot t_w \left[\left(a_{st.z.1} \right)^2 + \left(a_{st.z.2} \right)^2 + \left(a_{st.z.3} \right)^2 \right] = 0.021 m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029m^4$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \left(\frac{h_{s.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \left(\frac{h_{s.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.964$$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 t_s \varepsilon \sqrt{k_{\tau 1}}} = 0.886$ EC1-3-5 eq.5.6

Shear buckling coefficient, web with more than two longitudinal stiffeners

$\frac{a}{h_w} = 1.493$

SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[5 \left(\frac{h_w}{a} \right)^2 \sqrt[4]{\frac{I_{st.z.tot}}{\left[\min(t_s, t_w)^3 h_w \right]}}, \frac{2.1}{\min(t_s, t_w)} \sqrt[3]{\frac{I_{st.z.tot}}{h_w}} \right] = 4.782 \times 10^3$

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$$k_\tau := \begin{cases} 5.34 + 4 \left(\frac{h_{web}}{a} \right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \frac{I_{st.z.tot}}{\min(t_s, t_w)^3 h_{web}} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min(t_s, t_w)^3 h_{web}}} & \text{otherwise} \\ 4.1 + \frac{}{\alpha^2} \end{cases} = 1.131 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\overline{\lambda_w} = 1.2$ EC1993-1-3-5 ANM.2 5.1

$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \leq 31 \frac{\varepsilon \sqrt{k_\tau}}{\eta} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \varepsilon \sqrt{k_\tau}}, \lambda_{w1} \right) = 0.886$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 0.937$$

SS-EN 1993-1-5 5.3 Tabell 5.1, stiff end.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M1} := 1.00$ SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.429 \times 10^3 kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.851 \times 10^3 kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.999$ (contribution from two webs)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-1-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{tf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 569.539 \cdot mm$

$a_{tf.eff.neg} := h_n - \frac{h_b}{2} - z_{tp.eff.neg} = 708.261 \cdot mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \left| a_{tf.eff.neg} \right| \cdot f_y(t_d)}{\gamma_{M0}} \dots = 1.052 \times 10^4 kN \cdot m$
$+ \frac{A_{sf.eff.neg} \left| a_{sf.eff.neg} \right| \cdot f_y(t_b)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tf} := a \left[0.25 + \frac{1.6 b_d t_d^2 \cdot f_y(t_d)}{t_s \left(h_n + h_{wy} \right)^2 \cdot f_y(t_s)} \right] = 0.558m$

$c_{sf} := a \left[0.25 + \frac{1.6 b_b h_b^2 \cdot f_y(t_b)}{t_s \left(h_n + h_{wy} \right)^2 \cdot f_y(t_s)} \right] = 0.542m$

$V_{bf.Rd.cored} := \frac{b_d t_d^2 \cdot f_y(t_d)}{c_{tf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b h_b^2 \cdot f_y(t_b)}{c_{sf} \cdot \gamma_{M1} \cdot \gamma_n} = 325.381 \cdot kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cored} \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}} \right)^2 \right] & \text{if } M_{Ed} < M_{f.Rd.neg} = 0 \cdot kN \\ 0 & \text{otherwise} \end{cases}$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.429 \times 10^3 kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.999$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.018 m^4$

The flanges effective area:

$A_{eff.f.tot} := s_w W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.039 m^3 \cdot A_{d.eff} + A_b = 432.579 \cdot cm^2$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{sn.eff} := s_w \cdot a_{sn.eff} \cdot t_d \frac{f_y(t_d)}{2} = s_w \cdot a_{sn.eff} \cdot t_d \frac{f_y(t_d)}{2} \dots$

$M_{f.Rd} := W_{f.y.eff} \frac{f_y(t_d)}{\gamma_M} = 1.373 \times 10^4 kN \cdot m$

$z_{tp.f.eff.neg} := \frac{}{A_{eff.f.tot}} = 477.047 \cdot mm$

Results

$M_{pl.Rd} := M_{Rd} = 2.149 \times 10^4 kN \cdot m$

Moment capacity check when $q_{_M} = 151 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.996$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_{_V} = 154 \frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.999$

The Reduced stress method

-Changed side plate height to 790

Material

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 345MPa & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} = 1$

$E := 210GPa$

$\nu := 0.3$ $G := \frac{E}{2(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate:

$b_b := 16mm$ $h_b := 1000mm$ $A_b := b_b h_b = 0.016m^2$

Top plate

$t_{tp} := 40mm$ $b_{tp} := 3500mm$ $A_{tp} := b_{tp} t_{tp} = 0.035m^2$

Side plates

$t_s := 12mm$ $h_s := 790mm$ $A_s := h_s t_s = 9.48 \times 10^{-3} m^2$

Webs:

$t_w := 32mm$ $h_w := 1340mm$ $A_w := h_w t_w = 0.043m^2$

$h_{w,i} := 1250mm$ $h_{w,ij} := 482.8mm$

Top stiffeners

$t_{ts} := 16mm$ $b_{ts} := 140mm$ $A_{ts} := h_{ts} t_{ts} = 2.24 \times 10^{-3} m^2$

$s_{ts} := 350mm$ $n_{ts} := 9$

Web stiffeners

$t_{ws} := 12mm$ $b_{ws} := 120mm$ $A_{ws} := h_{ws} t_{ws} = 1.44 \times 10^{-3} m^2$

$s_{ws} := 440mm$ $n_{ws} := 3$

Whole bridge

$L_{bridge} := 18m$ $h_b := \frac{h_b}{2} + h_{w,ij} + h_s + t_d = 1.291\,m$ $\eta_b := 2in$

$I_{yy} := 0.0163m^4$ From grasshopper

$V_{gr} := 0.7564\,kn$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_v := 120 \frac{kN}{m^2}$

$Q := q_v b_{tp} = 420 \frac{kN}{m}$

$M_{Ed} := \frac{Q\, L_{bridge}^2}{8} = 1.701 \times 10^4\,kN\,m$

$V_{Ed} := \frac{Q\, L_{bridge}}{2} = 3.78 \times 10^3\,kN$

Top plate:

$\sigma_{x,Ed,tp} := \frac{M_{Ed}}{I_{yy}} \left(h_w - \frac{t_d}{2} - y_{tp} \right) = 262.139\,MPa$ $\sigma_{z,Ed,tp} := 0$

$S_{yy} := \frac{t_d b_{tp}}{n_{ts}+1} \left(h_w - \frac{t_d}{2} - y_{tp} \right) = 1.747 \times 10^{-3} m^3$

$\tau_{Ed,tp} := \frac{V_{Ed} S_{yy}}{I_{yy} \min(t_d, t_{ts})} = 15.446\,MPa$

Side plates

$\sigma_{x,Ed,sp} := \frac{M_{Ed}}{I_{yy}} \left(h_w - t_d - y_{sp} \right) = 259.752\,MPa$

$\sigma_{z,Ed,sp} := \frac{q_v}{n_{ts}} = 17.5\,MPa$

$S_{sp} := \left(h_w - t_d - y_{sp} \right) \left(t_s \frac{h_w - t_d - y_{sp}}{2} + \frac{t_d b_{tp}}{n_{ts}+1} \left(h_w - \frac{t_d}{2} - y_{sp} \right) \right) = 0.016\,m^3$

$S_{ps} := \left(t_{ts} \frac{h_{ts}}{2} + t_s \frac{t_s h_{ts}}{n_{ts}+1} \right) \left(h_w - t_d - \frac{h_{ts}}{4} - y_{sp} \right) = 2.292 \times 10^{-3} m^3$

$\tau_{Ed,sp} := \frac{V_{Ed} S_{ps}}{I_{yy} t_s}$

$\tau_{Ed,sp} = 15.193\,MPa$

Group using yield stress from FE-software

$\sigma_{V,UNIT} := 1 \frac{\frac{MPa}{kN}}{m^2}$ From a linear stress analysis in Softalk

4.2449

$\sigma_{V} := \sigma_{V,UNIT} q_v = 434.4\,MPa$

$\alpha_{ult} := \frac{f_y(t)}{\sigma_V} = 0.817$

$\alpha_{cr,UNIT} := 353.99 \frac{kN}{m^2}$ From a buckling eigenvalue analysis in Softalk

$\alpha_{cr} := \frac{\alpha_{cr,UNIT}}{q_v} = 2.117$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.621$ Slenderness for all parts

Top plate check

$\alpha_{tp} := 1$

$\rho_{p,tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055(3+\alpha_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E,tp} := \frac{\pi^2 E t_d^2}{12(1-\nu^2) b_{tp}^2} = 1.589\,MPa$

$I_{yy,tp} := \frac{b_{tp} t_d^3}{12(1-\nu^2)} = 3.205 \times 10^{-7} m^4$

$y_{tp,el,tp} := \frac{A_{tp} \frac{t_d}{2} + n_{ts} A_{ts} \left(t_d + \frac{h_{ts}}{2} \right)}{A_{tp} + n_{ts} A_{ts}} = 32.411\,mm$

$I_{el,tp} := \frac{b_{tp} t_d^3}{12} + A_{tp} \left(y_{tp,el,tp} - \frac{t_d}{2} \right)^2 + n_{ts} \left[\frac{t_{ts} h_{ts}^3}{12} + A_{ts} \left(t_d + \frac{h_{ts}}{2} - y_{tp,el,tp} \right)^2 \right] = 1.052 \times 10^{-3} m^4$

$\gamma_{tp} := \frac{I_{el,tp}}{I_{yy,tp}} = 328.143$

$\delta_{tp} := \frac{n_{ts} A_{ts}}{A_{tp}} = 0.576$

$\alpha_{tp} := \max\left(\frac{\gamma_{tp}}{\gamma_{tp}}, 0.5 \right) = 0.571$ $\sqrt{\gamma_{tp}} = 4.256$

$k_{\sigma,tp,tp} := \begin{cases} \frac{2\left[(1+\alpha_{tp})^2 + \gamma_{tp} - 1 \right]}{\alpha_{tp}^2 (\delta_{tp}+1)(1+\delta_{tp})} & \text{if } \alpha_{tp} \sqrt{\gamma_{tp}} < 1.09 \\ \frac{4(1+\sqrt{\gamma_{tp}})}{(\delta_{tp}+1)(1+\delta_{tp})} & \text{if } \alpha_{tp} \sqrt{\gamma_{tp}} \ge 1.09 \end{cases} = 639.127$

$\sigma_{cr,tp,tp} := k_{\sigma,tp,tp} \sigma_{E,tp} = 990.256\,MPa$

Column-like behavior [4.5.3]

$b_{1,eff,tp} := \frac{1 - \eta_{tp}}{1 - \eta_{tp}} b_{tp} = 0.175\,m$

$b_{2,eff,tp} := \frac{2}{5 - \eta_{tp}} b_{tp} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{el,tp} := b_{1,eff,tp} + b_{2,eff,tp} = 0.35\,m$

$y_{tp,el,tp} := \frac{t_d b_{el,tp} \frac{t_d}{2} + n_{ts} A_{ts} \left(t_d + \frac{h_{ts}}{2} \right)}{t_d b_{el,tp} + n_{ts} A_{ts}} = 0.134\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{el,tp} := \frac{b_{el,tp} t_d^3}{12} + b_{el,tp} t_d \left(y_{tp,el,tp} - \frac{t_d}{2} \right)^2 + \frac{t_{ts} h_{ts}^3}{12} + A_{ts} \left(t_d + \frac{h_{ts}}{2} - y_{tp,el,tp} \right)^2$ Combined moment of inertia

$\sigma_{cr,el,tp} := \frac{\pi^2 E I_{el,tp}}{A_{el,tp} s_{ts}^2} = 1.026 \times 10^3\,MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr,tp} = \sigma_{cr,el}$

$\sigma_{cr,c,tp} := \sigma_{cr,el,tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{el,tp}}{A_{el,tp}}} = 0.045\,m$

$s_{tp} := \min\left(\eta_{el,tp} - \eta_{tp} t_d + \frac{b_{tp}}{2} - y_{tp,el,tp} \right) = 0.846\,m$

$\alpha_{tp,tp} := 0.49 + \frac{0.09}{\frac{s_{tp}}{i_{tp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5 \left[1 + \alpha_{tp,tp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 0.816$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1 \right) = 0.744$

Interpolation

$\eta_{tp} := \min\left(\max\left(\frac{\sigma_{cr,tp,tp}}{\sigma_{cr,c,tp}} - 1.0, 0 \right), 1 \right) = 0$

$\rho_{c,tp} := \begin{cases} \left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp} \left(2 - \eta_{tp} \right) + \chi_{tp} \right] & \text{if } \frac{\left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp} \left(2 - \eta_{tp} \right) + \chi_{tp} \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp} \left(2 - \eta_{tp} \right) + \chi_{tp} \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.744$

Shear

$\eta_s := 1.2$ 7.1 (2)

$\lambda_w := \begin{cases} \eta_s & \text{if } \lambda_p < \frac{0.83}{\eta_s} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_s} \le \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$

$\frac{\rho_{c,tp} \gamma_{M0}}{\gamma_{M1}} = 0.608$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{f_y(t_d)} z + \sigma_{max} \right] - 1$

$\psi_{tp,x} := \frac{\sigma(h_w - t_d - h_s)}{\sigma(h_w - t_d)} = -0.452$

$\sigma_{x,tp,tp} := \frac{M_{Ed}}{I_{yy}} \left(h_w - t_d - y_{tp} \right) = 259.752\,MPa$

$\sigma_{E,tp,x} := \frac{\pi^2 E t_d^2}{12(1-\nu^2) b_{tp}^2} = 43.793\,MPa$ $\frac{M_{cr,tp}}{W_{el,tp}} = 17.336\,MPa$

$\psi_{tp,1,x} := \max(1, \psi_{tp,x}, -1) = 2.548$ $\psi_{tp1} := \frac{\sigma_{x,tp,tp}}{\sigma_{x,tp}} = -0.452$

$\rho_{p,tp,tp} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{\lambda_p - 0.055(3+\psi_{tp,1,x})}{\lambda_p^2} & \text{if } \lambda_p \ge 0.673 \end{cases} = 1$

$k_{\sigma,tp,x} := \begin{cases} 1 & \text{if } \psi_{tp,x} \ge 1 \\ \frac{8.2}{1.05 + \psi_{tp,x}^2} & \text{if } 1 > \psi_{tp,x} > 0 \\ 7.81 & \text{if } \psi_{tp,x} = 0 \\ 7.81 - 6.29\,\psi_{tp,x} + 9.78\,\psi_{tp,x}^2 & \text{if } 0 > \psi_{tp,x} > -1 \\ 23.9 & \text{if } \psi_{tp,x} = -1 \\ 5.98\left(1 - \psi_{tp,x}\right)^2 & \text{if } -1 > \psi_{tp,x} > -3 \\ 35.87 & \text{if } \psi_{tp,x} < -3 \end{cases} = 12.647$

$\sigma_{cr,tp,tp,x} := \sigma_{E,tp,x} k_{\sigma,tp,x} = 553.849\,MPa$

Column-like behaviour

$\sigma_{cr,c,tp,x} := \frac{\pi^2 E t_d^2}{12(1-\nu^2) s_{ts}^2} = 6.833\,MPa$

$\alpha_{tp} = 0.21$

$\phi_{tp} := 0.5 \left[1 + \alpha_{tp} \left(\lambda_p - 0.2 \right) + \lambda_p^2 \right] = 0.737$

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1 \right) = 0.882$

Interpolation

$\eta_{tp,x} := \min\left(\max\left(\frac{\sigma_{cr,tp,tp,x}}{\sigma_{cr,c,tp,x}} - 1.0, 0 \right), 1 \right) = 1$

$\rho_{c,tp,x} := \begin{cases} \left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp,x} \left(2 - \eta_{tp,x} \right) + \chi_{tp} \right] & \text{if } \frac{\left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp,x} \left(2 - \eta_{tp,x} \right) + \chi_{tp} \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp,x} \left(2 - \eta_{tp,x} \right) + \chi_{tp} \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

Z-direction

$\psi_{tp,z} = 1$

$\alpha_{p,tp} = 0.34$ $\lambda_{p0,tp} = 0.7$

$\phi_{p,tp} := 0.5 \left[1 + \alpha_{p,tp} \left(\lambda_p - \lambda_{p0,tp} \right) + \lambda_p^2 \right] = 0.797$

$\rho_{p,tp,z} := \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{p,tp} + \sqrt{\phi_{p,tp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma,p,tp,z} = 4$

$\sigma_{E,tp,z} := \frac{\pi^2 E t_d^2}{12(1-\nu^2) s_{ts}^2} = 6.833\,MPa$

$\sigma_{cr,tp,tp,z} := k_{\sigma,p,tp,z} \sigma_{E,tp,z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{cr,c,tp,z} := \frac{\pi^2 E t_d^2}{12(1-\nu^2) s_{ts}^2} = 43.793\,MPa$

Interpolation

$\eta_{tp,z} := \min\left(\max\left(\frac{\sigma_{cr,tp,tp,z}}{\sigma_{cr,c,tp,z}} - 1.0, 0 \right), 1 \right) = 0$

$\rho_{c,tp,z} := \begin{cases} \left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp,z} \left(2 - \eta_{tp,z} \right) + \chi_{tp} \right] & \text{if } \frac{\left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp,z} \left(2 - \eta_{tp,z} \right) + \chi_{tp} \right] \gamma_{M0}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{p,tp,tp} - \chi_{tp} \right) \eta_{tp,z} \left(2 - \eta_{tp,z} \right) + \chi_{tp} \right] \gamma_{M0}}{\gamma_{M1}} \ge 1 \end{cases} = 0.882$

Top plate stiffeners - check

x-direction

$$\rho_{p,c,sys} = \begin{cases} 1 & \text{if } \lambda_p < 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{sys,x} = \frac{\sigma 2 \left(\lambda_y - t_f - t_w\right)}{\sigma 1 \left(\lambda_y - t_f\right)} = 0.743$$

$$\sigma_{E,sys,x} = \frac{\pi^2 E {t_{sx}}^2}{12 \left(1 - \nu^2\right) {\lambda_{sx}}^2} = 2.479 \times 10^3 \, MPa$$

$$k_{\sigma,sys,x} = \begin{cases} 4 & \text{if } \psi_{sys,x} \geq 1 \\ \frac{8.2}{1.05 + \psi_{sys,x}} & \text{if } 1 > \psi_{sys,x} > 0 \\ 7.81 & \text{if } \psi_{sys,x} = 0 \\ 7.81 - 6.29 \psi_{sys,x} + 9.78 \psi_{sys,x}^2 & \text{if } 0 > \psi_{sys,x} > -1 \\ 23.9 & \text{if } \psi_{sys,x} = -1 \\ 5.98 \left(1 - \psi_{sys,x}\right)^2 & \text{if } -1 > \psi_{sys,x} > -3 \\ 35.68 & \text{if } \psi_{sys,x} < -3 \end{cases} = 5.134$$

$$\sigma_{cr,sys,x} = k_{\sigma,sys,x} \sigma_{E,sys,x} = 1.273 \times 10^4 \, MPa$$

Column-like behaviour

$$\sigma_{cr,c,sys,x} = \frac{\pi^2 E {i_{sx}}^2}{\left(1 - \nu^2\right) {\lambda_{sx}}^2} = 12.147 \, MPa$$

$$\chi_{sys} = \chi_{sp}$$

Interpolation

$$\xi_{sys,x} = \min\left(\max\left(\frac{\sigma_{cr,sys,x}}{\sigma_{cr,c,sys,x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c,sys,x} = \begin{cases} \left[\left[\rho_{p,c,sys} - \chi_{sys}\right] \xi_{sys,x} \left(2 - \xi_{sys,x}\right) - \left[\rho_{p,c,sys} - \chi_{sys}\right] \xi_{sys,x}^2 + \chi_{sys}\right]^{m_{sh}} & \text{if } \frac{\left[\left[\rho_{p,c,sys} - \chi_{sys}\right] \xi_{sys,x} \left(2 - \xi_{sys,x}\right) - \left[\rho_{p,c,sys} - \chi_{sys}\right] \xi_{sys,x}^2 + \chi_{sys}\right]^{m_{sh}}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left[\rho_{p,c,sys} - \chi_{sys}\right] \xi_{sys,x} \left(2 - \xi_{sys,x}\right) - \left[\rho_{p,c,sys} - \chi_{sys}\right] \xi_{sys,x}^2 + \chi_{sys}\right]^{m_{sh}}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sys,x} f_y}{\gamma_{M1}}}\right)^2 + 3 \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w f_y}{\gamma_{M1}}}\right)^2 = 0.909$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sys,x} f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sys,x} f_y}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sys,x} f_y}{\gamma_{M1}}}\right)\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sys,x} f_y}{\gamma_{M1}}}\right) + 3 \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w f_y}{\gamma_{M1}}}\right)^2 = 0.582$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sys,x} f_y}{\gamma_{M1}}}\right)^2 + 3 \left(\frac{\tau_{Ed,sp}}{\frac{\chi_w f_y}{\gamma_{M1}}}\right)^2 = 0.339$$

EWM - Changed side plate height

This chapter covers the EWM when changing the side plate height to 870 mm

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s,so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d,0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s,so}} = 350\,mm$

Side plate: $h_s := 870mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su}\cdot\cos\left(21.12\cdot\frac{\pi}{180}\right) = 111.939\,mm$

Total height: $h_{ls} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 1.371\times10^{3}\,mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 1.345\times10^{3}\,mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Area

$A_d := t_d\cdot b_{uarameter}$ $A_s := 2t_s h_s$ $A_w := 2t_w h_w$ $A_b := t_b b_b$

$A_{so} := n_{so} t_{so} b_{so}$ $A_{su} := n_{su} t_{su} b_{su}$

$E := 210GPa$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.133\,m^2$ tab. 3.1

$A_c := A_{tot} + A_d = 0.055\,m^2$ ok

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 345MPa & \text{if } 40mm < t \le 80mm \end{cases}$

Changed center of gravity, z_{sp}

$$z_{sp} := \frac{A_d\frac{t_d}{2} + A_{so}\left(t_d + \frac{h_{so}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) + \ldots}{A_{tot}} + \ldots = 592.696\,mm$$

$$\ldots + A_b\left(t_d + h_s + h_{wy}\right) + A_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right]$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\,mm$

Stiffeners overlunder:

$$I_{y.st.so} := n_{so}\frac{t_{so}\,h_{so}^{\,2}}{12} + n_{so}\,t_{so}\,h_{so}\left(t_d + \frac{h_{so}}{2} - z_{sp}\right)^2 = 5.332\times10^{-3}\,m^4$$

$I_{y.st.so.1} := 6\cdot1.506\cdot10^{-6}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su}\,h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 + \ldots = 2.752\times10^{-3}\,m^4$$

$$\ldots + 2t_{su}\,h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2 \ldots$$

$$\ldots + 2t_{su}\,h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{sp}\right]^2$$

Web:

$I_{y.w.1} := 2\cdot0.000312m^4$ from FEM

$$I_{y.w} := I_{y.w.1} + 2t_w h_w\left[h_{ls} - \frac{t_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{sp}\right]^2 = 9.614\times10^{-3}\,m^4$$

Deck:

$$I_{y.d} := \frac{b_d\,t_d^{\,3}}{12} + t_d\,b_d\left(\frac{t_d}{2} - z_{sp}\right)^2 = 0.012m^4$$

Bottom flange:

$$I_{y.b} := \frac{b_b\,t_b^{\,3}}{12} + t_b\,b_b\left(h_{ls} - z_{sp} - \frac{t_b}{2}\right)^2 = 9.489\times10^{-3}\,m^4$$

Side plate:

$$I_{y.s} := 2\frac{t_s\,h_s^{\,3}}{12} + 2t_s\,h_s\left(t_d + \frac{h_s}{2} - z_{sp}\right)^2 = 1.772\times10^{-3}\,m^4$$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.041\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355\,MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{sp}}\,z - \sigma_{max}$

$\sigma_{1.neg}(\sigma_1,\sigma_2) := \max(\sigma_{neg}(\sigma_1),\sigma_{neg}(\sigma_2))$

$\sigma_{2.neg}(\sigma_1,\sigma_2) := \min(\sigma_{neg}(\sigma_1),\sigma_{neg}(\sigma_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{sp}}\,z + \sigma_{max}$

$\sigma_{1.pos}(\sigma_1,\sigma_2) := \max(\sigma_{pos}(\sigma_1),\sigma_{pos}(\sigma_2))$

$\sigma_{2.pos}(\sigma_1,\sigma_2) := \min(\sigma_{pos}(\sigma_1),\sigma_{pos}(\sigma_2))$

Buckling factor

$\psi_{neg}(\sigma_1,\sigma_2) := \frac{\sigma_{2.neg}(\sigma_1,\sigma_2)}{\sigma_{1.neg}(\sigma_1,\sigma_2)}$ $\psi_{pos}(\sigma_1,\sigma_2) := \frac{\sigma_{2.pos}(\sigma_1,\sigma_2)}{\sigma_{1.pos}(\sigma_1,\sigma_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1-\psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

$$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \ge -3 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$$k_{\sigma.4.2j}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(b_p,h,t) := \frac{\frac{b_p}{t}}{28.4\cdot\varepsilon(t)\cdot\sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p,\psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left(\frac{\lambda_p - 0.055\max(3+\psi,0)}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2) Internal parts

$$\rho_{4.3}(\lambda_p,\psi) := \begin{cases} 1.0 & \text{if } \lambda_p \le 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^{\,2}}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10\,mm$

$z_2 := t_d + h_s = 880\,mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.493$

$$Condition := \begin{cases} 62\cdot\varepsilon(f_y)\cdot(1-\psi_s)\cdot\sqrt{-\psi_s} & \text{if } \psi_s \le -1 \\ \frac{42\cdot\varepsilon(f_y)}{0.67 + 0.33\cdot\psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} \quad = \text{"Class4"}$$

$b_{\sigma.4.1}(\psi_s) = 13.289$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(b_{\sigma.4.1}(\psi_s), h_w, t_s) = 0.861$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 0.976$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{sp} = 287.304\,mm$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 582.696\,mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 568.546\,mm$

$h_{s.eff.c1} := 0.4\,h_{s.eff.c} = 227.418\,mm$

$h_{s.eff.c2} := 0.6\,h_{s.eff.c} = 341.127\,mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 855.849\,mm$

$A_{s.eff.c} := 2\,h_{s.eff.c}\,t_s = 1.365\times10^{4}\,mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t})\,t_s = 2.054\times10^{4}\,mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s)\frac{E\cdot\pi^2\cdot t_s^{\,2}}{12\left(1-\nu^2\right)h_{s.t}^{\,2}} = 479.855\,MPa$ EC1993-1-5 eq A.1

Stiffeners

$t_{so} := t_d = 10\,mm$

$h_w := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{mg}(z_2)}{\sigma_{mg}(z_1)} = 0.76$

$k_{\sigma.4.2}(\psi_{so}) = 0.526$ Higher stress close to support

Class4_ := "Class4" if $\frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon \cdot \sqrt{k_{\sigma.4.2}(\psi_{so})}$ = "Lower than class 4" "Lower than class 4" otherwise

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2}(\psi_{so}), h_{so}, t_{so}\right) = 0.522$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_d) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{3}\ mm^2$

Deck

$\frac{h_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

Class4_ := "Class4" if $\frac{h_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}}$ = "Class4" "Lower than class 4" otherwise

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\ m$ $\psi_d := \psi_{mg}(z_d, z_d) = 1$

Remark $b_{so} := t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(b_{\sigma.d}, z_{c.so}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$x_{so.eff} := \rho_d \cdot x_{c.so} = 321.461\ mm$

Areas:

$A_{tot} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.31 \times 10^{4}\ mm^2$

$A_{c.eff.loc} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.053 \times 10^{4}\ mm^2$

More than three longitudinal stiffeners -> $k_{\sigma p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{5}\ mm^4$

$y_{tp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411\ mm$

$I_d := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{8}\ mm^4$

$\gamma := \frac{I_d}{I_p} = 328.143 \qquad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02\ m^2$ EC1993-1-5A.1 (2)

Critical buckling stress for orthotropic plate:

Remark $\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^2}{12 \cdot (1 - \nu^2) \cdot b_d^2} = 990.256\ MPa$ EC1993-1-5 eq.A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{p.c} := \begin{cases} \frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}} & \text{if } \alpha \le \sqrt{\frac{1}{\beta_{A.c}}} \\ \frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}} \cdot \left(0.7 + 0.3 \cdot \sqrt{\frac{1}{\beta_{A.c}}}\right) & \text{otherwise} \end{cases} = 639.127$

$\lambda_{p.c} = 0.584$

$\rho := \rho_{4.2}(\lambda_{p.c}, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_{p.c}, \psi) \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d = 5.375 \times 10^{4}\ mm^2$

Bucking strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^{4}\ kN$

Column like behaviour

$y_{tp.sl.c} := \frac{n_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot h_{so} + n_{so} \cdot t_d} = 34.268\ mm$

$I_{sl.c} := \frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2 + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{-6}\ m^4$

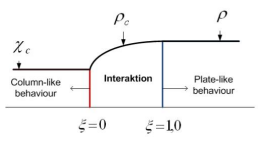
Bucking resistance stiffener

Remark $t_{so} \cdot h_{so} + n_{so} \cdot t_d = 5.74 \times 10^{-3}\ m^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + n_{so.eff} \cdot t_d = 5.455 \times 10^{-3}\ m^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical bucking stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot s^2} = 1.026 \times 10^{3}\ MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sl})}{\sigma_{cr.sl}}} = 0.573$

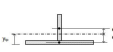
Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\ mm$

$e_c := \max\left(\left|y_{tp.sl} - \frac{t_d}{2}\right|, \left|\left(t_d + \frac{h_{so}}{2}\right) - y_{tp.sl}\right|\right) = 47.589\ mm$



$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{c.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero -> column like behviour

Interaktion mellan de två olika beteendena(column/plate bucking):

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{axst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{mg}\left(t_d + \frac{t_d}{2}\right) + A_{so} \cdot \sigma_{mg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.637 \times 10^{4}\ kN$

$\sigma_{com.Ed} := \frac{F_{axst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -333.558\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} = 281.032\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{sl})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1 (4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{so} \cdot s_{so.eff} \cdot t_d = 0.032\ m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{so} + A_b + A_{sl.eff} = 0.13\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ &\left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{so} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - x_{sol}\right) - \frac{h_{so.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_n - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 606.546\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right. \\ \left. + \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2\right] \dots \right. \\ \left. + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right]\right] = 1.817 \times 10^{-3}\ m^4$

$I_{y.d.eff} := \frac{n_{so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{so} \cdot s_{so.eff} \cdot t_d \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 0.012\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s.eff} + I_{y.d.eff} = 0.042\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{d.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^{4}\ mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_d) \cdot A_{cl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.836 \times 10^{3}\ mm^2$

$A_{so.eff.neg}$ Loaded stiffener in tension -> no reduction

A_b Bottom flange in tension

Tension -> No reduction

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is applied and the area above is used.

$A_{so.red}$ NO REDUCTION OF SIDE PLATE is in class 4.

$A_{so.red} := \begin{cases} \text{Solve Block} & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 432.173\ cm^2$

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{so.eff} + A_w + A_{so} + A_b = 1.206 \times 10^{-2}\ m^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.916\ mm$

$z_{tp.eff.red} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) \dots \\ &+ s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_d \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) \dots \\ &+ t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ &+ A_{so} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - x_{sol}\right) - \frac{h_{so.y}}{2}\right] + A_b \cdot \left(h_n - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 611.043\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so} \cdot s_{so.eff} \cdot t_{d.eff}^3}{12} + t_d \cdot s_{so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 0.012\ m^4$

Remark $t_{sol} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{sol} = 7.206\ mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 5.718 \times 10^{-3}\ m^4$

$I_{y.st.su.1.fin} = 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots = 2.581 \times 10^{-3}\ m^4$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - x_{sol}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - x_{sol}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$
$+ 2 \cdot t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - x_{sol}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$

Web:

$I_{y.w.1.fin} = 2 \cdot 0.000312\ m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[h_n - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 9.001 \times 10^{-3}\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{h_b \cdot t_b^3}{12} + t_b \cdot h_b \cdot \left(h_n - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 9.043 \times 10^{-3}\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \right. \\ \left. + \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2\right] \dots \right. \\ \left. + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right]\right] = 1.847 \times 10^{-3}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 4 \times 10^{10}\ mm^4$

Load

Load capacity:

$q_M := 164 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_{tf} = 574 \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 2.325 \times 10^4 \, kN \cdot m$

$q_V := 152 \frac{kN}{m^2}$ The load capacity is q_v when checking the shear capacity.

$Q_V := q_V \cdot b_{tf} = 532 \frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.788 \times 10^3 \, kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.ltin}}{z_{tp.eff.neg}} = 0.065 \, m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_fl)}{\gamma_{M1}} = 2.324 \times 10^4 \, kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 1$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 S.1 ANM.2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$h_{webb} := 2 \cdot 15 \cdot t_{sl} \cdot t_w = 0.293 m$

$\alpha_w := \min\left(\frac{h_{webb}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{h_b}{2} + s_{sl.fl} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ss}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{ss}}{2} = 0.503 m$

$a_{st.z.2} := a_{st.z.1} + h_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913 m$

$a_{st.z.3} := a_{st.z.2} + h_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324 m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 \cdot t_{ss} \cdot h_{ss} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3} \, m^4$ (two stiffeners)

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$a_{webb} := \frac{h_b}{2} + s_{sl.fl} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ss}}{2} = 0.525 m$

$a_{webb} := a_{st.z.1} + h_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935 m$

$a_{webb} := a_{st.z.2} + h_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346 m$

Moment of inertia of 15st of the web z-direction

$I_{stw.z} := 2 h_{webb} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \, m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029 m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ \text{otherwise} \end{cases} = 6.072$

$k_{\tau 2} := \max\left[9 \left(\frac{h_w}{a}\right)^2 \sqrt[3]{\frac{I_{st.z.tot}}{\left(\min\left(t_f, t_w\right)\right)^3 \cdot h_w}}, \frac{2.1}{\min\left(t_f, t_w\right)} \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

$\lambda_{w1} := \frac{37.4 \cdot t_w \cdot \varepsilon(t_w)}{\sqrt{k_{\tau 1}}} = 0.951$ EC1-3-5 eq.5.6

SS-EN 1993-25 A3 (5.10) (2) gir:

$k_\tau := \begin{cases} 5.34 + 4 \left(\frac{h_{webb}}{a}\right)^2 + k_{\tau 2} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \frac{I_{st.z.tot}}{\min\left(t_f, t_w\right)^3 \cdot h_{webb}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_f, t_w\right)^3 \cdot h_{webb}}} & \text{otherwise} \end{cases} = 1.066 \times 10^3$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\overline{\eta}_w := 1.2$ EC1993-1-3-S ANM.2 5.1

$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{webb}}{\min\left(t_f, t_w\right)} \leq 31 \frac{\varepsilon(t_f)}{\eta} \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w) \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.951$ SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 0.873$ SS-EN 1993-1-5 5.3 Tabell 5.1, stiff end.

Kapacity of the web

$\gamma_{M0} := 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$ SB3 "Tillrande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_w) \cdot h_{webb} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.405 \times 10^3 \, kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.788 \times 10^3 \, kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.995$ (contribution from two webs)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{webb} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{tf.eff.neg} := z_{tp.eff.neg} - \frac{t_{fl}}{2} = 606.043 mm$

$z_{tf.eff.neg} := h_{tb} - \frac{t_b}{2} - z_{tp.eff.neg} = 751.757 mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \left| z_{tf.eff.neg} \right| \cdot f_y(t_{fl})}{\gamma_{M0}} \ldots = 1.119 \times 10^4 \, kN \cdot m$
$+ \frac{A_{tf.eff.neg} \left| z_{tf.eff.neg} \right| \cdot f_y(t_b)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tf} := a \left[0.25 + \frac{1.6 \cdot b_{fl} \cdot t_{fl}^2 \cdot f_y(t_{fl})}{t_s \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.551 m$

$c_{tf} := a \left[0.25 + \frac{1.6 \cdot b_{fl} \cdot t_{fl}^2 \cdot f_y(t_{fl})}{t_s \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.537 m$

$V_{bf.Rd.cred} := \frac{b_{sl} \cdot t_{sl}^2 \cdot f_y(t_{sl})}{\gamma_{sl} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{h_b \cdot t_b^2 \cdot f_y(t_b)}{\gamma_{sl} \cdot \gamma_{M1} \cdot \gamma_n} = 328.867 kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cred} \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} = \frac{\eta \cdot f_y(t_w) \cdot h_{webb} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{webbar} := V_{bw.Rd} + V_{bf.Rd} = 2.405 \times 10^3 kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.995$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.ltin} := I_{y.sl.eff.ltin} + I_{y.b.ltin} = 0.021 \cdot m^4$

The flanges effective area:

$A_{sl.eff.tot} := n_{sn.sl.f} \cdot t_{sl} \cdot s_{sn.sl.f} \cdot t_{sl.eff} + n_{sn} \cdot s_{sn.sl} \cdot t_{sl.eff} + A_b = 432.579 \cdot cm^2$

$z_{tp.f.eff.neg} := \frac{\begin{matrix} s_{sn.sl.f} \cdot t_{sl} \cdot \frac{t_{sl}}{2} + n_{sn} \cdot s_{sn.sl} \cdot t_{sl.eff} \cdot \left(\frac{t_{sl.eff}}{2}\right) \ldots \\ + n_{sn} \cdot s_{sn} \cdot t_{sl.eff} \cdot \left(\frac{t_{sl.eff}}{2}\right) + A_b \left(h_b - \frac{t_b}{2}\right) \end{matrix}}{A_{sl.f.tot}} = 506.637 mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.ltin}}{z_{tp.f.eff.neg}} = 0.041 \, m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_{fl})}{\gamma_M} = 1.461 \times 10^4 \, kN \cdot m$

Results

$M_{pl.Rd} := M_{f.Rd} = 2.324 \times 10^4 \, kN \cdot m$

Moment capacity check when $q_M = 164 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 1$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 152 \frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 V_{bw.Rd}} = 0.995$

The Reduced stress method

-Changed side plate height to 870 mm

Material

$f_y(t) := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ 335\,MPa & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} = 1$

$E = 210\,GPa$

$\nu = 0.3$ $G = \frac{E}{2(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate:

$t_b = 16\,mm$ $b_b = 1000\,mm$ $A_b = b_b t_b = 0.016\,m^2$

Top plate

$t_d = 10\,mm$ $b_d = 3500\,mm$ $A_d = b_d t_d = 0.035\,m^2$

Side plates

$t_s = 12\,mm$ $h_s = 870\,mm$ $A_s = h_s t_s = 0.01\,m^2$

Webs:

$t_w = 12\,mm$ $h_w = 1340\,mm$ $A_w = h_w t_w = 0.016\,m^2$

$h_{w.k} = 1258\,mm$ $h_{w.y} = 482.8\,mm$

Top stiffeners

$t_{td} = 16\,mm$ $b_{td} = 140\,mm$ $A_{td} = b_{td} t_{td} = 2.24 \times 10^{-3}\,m^2$

$s_{td} = 350\,mm$ $n_{td} = 9$

Web stiffeners:

$t_{tw} = 12\,mm$ $b_{tw} = 120\,mm$ $A_{tw} = b_{tw} t_{tw} = 1.44 \times 10^{-3}\,m^2$

$s_{tw} = 440\,mm$ $n_{tw} = 3$

Whole bridge

$L_{bridge} = 18\,m$ $h_h = \frac{h_b}{2} + h_{w.y} + h_s + t_d = 1.371\,m$ $n_b = 2\,m$

$I_{yd} = 0.041028\,m^4$ From grasshopper

$y_{gd} = 0.778308\,m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_1 = 118\,\frac{kN}{m}$

$Q = q_1 n_b = 413\,\frac{kN}{m}$

$M_{Ed} = \frac{Q L_{bridge}^2}{8} = 1.673 \times 10^4\,kN\,m$

$V_{Ed} = \frac{Q L_{bridge}}{2} = 3.717 \times 10^3\,kN$

Top plate

$\sigma_{x.Ed.up} = \frac{M_{Ed}}{I_{yd}} \left(h_h - \frac{t_d}{2} - y_{gd} \right) = 239.512\,MPa$ $\sigma_{x.Ed.up} = 0$

$S_{yd} = \frac{t_d b_d}{n_{tw}+2} \left(h_h - \frac{t_d}{2} - y_{gd} \right) = 1.869 \times 10^{-3}\,m^3$

$\tau_{Ed.up} = \frac{V_{Ed} S_{yd}}{t_w \min(t_s, t_{td})} = 14.113\,MPa$

Side plates:

$\sigma_{x.Ed.sp} = \frac{M_{Ed}}{I_{yd}} \left(h_h - t_d - y_{gd} \right) = 237.473\,MPa$

$\sigma_{z.Ed.sp} = \frac{Q}{n_b} = 17.208\,MPa$

$S_{ys} = \left(h_h - t_d - y_{gd} \right) t_s \frac{\left(h_h - t_d - y_{gd} \right)}{2} + \frac{t_d b_d}{n_{tw}+2} \left(h_h - \frac{t_d}{2} - y_{gd} \right) = 0.017\,m^3$
$+ \frac{n_{tw}}{n_{tw}+2} t_{tw} t_s \left(h_h - t_d - \frac{h_w}{2} - y_{gd} \right)$

$S_{yw} = \frac{\left(\frac{t_w h_w}{2} + 2 \frac{t_d b_d}{n_{tw}+2} \right) \left(h_h - t_d - \frac{h_w}{4} - y_{gd} \right)}{+ \frac{t_d b_d}{n_{tw}+2} \left(h_h - \frac{t_d}{2} - y_{gd} \right)} = 2.455 \times 10^{-3}\,m^3$

Tim ntato stiffenerrs

$\tau_{Ed.sp} = \frac{V_{Ed} S_{yw}}{2 t_w I_{yd}} = 13.899\,MPa$

$\sigma_{x.Ed.sp} = \frac{M_{Ed}}{I_{yd}} \left(h_h - t_d - y_{gd} \right) = 237.473\,MPa$

Governing fffendemess from FE-software

$\sigma_{V.UNIT} = 2.85\,\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Solfstk

4.2449

$\sigma_V = \sigma_{V.UNIT} q_1 = 336.3\,MPa$

$\alpha_{ult} = \frac{f_y(t_d)}{\sigma_V} = 1.056$

$\alpha_{cr.UNIT} = 238.62\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Solfstk

$\alpha_{cr} = \frac{\alpha_{cr.UNIT}}{q_1} = 2.022$

$\lambda_y = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.723$ Slenderness for all parts

Top plate check

$\psi_{sp} = 1$

$\rho_{p.sp} = \begin{cases} 1 & \text{if } \lambda_y < 0.673 \\ \frac{\lambda_y - 0.055(3+\psi_{sp})}{\lambda_y^2} & \text{if } \lambda_y > 0.673 \end{cases} = 0.963$

Plate-like behaviour [A.1]

$\sigma_{E.sp} = \frac{\pi^2 E t_d^2}{12(1-\nu^2) b_s^2} = 1.549\,MPa$

$I_{yd} = \frac{b_d t_d^3}{12(1-\nu^2)} = 3.205 \times 10^{-7}\,m^4$

$y_{gd.sp} = \frac{A_d \frac{t_d}{2} + A_w h_w \left(t_d + \frac{h_w}{2} \right)}{A_d + n_w A_w} = 32.411\,mm$

$I_{y.sp} = \frac{b_d t_d^3}{12} + A_d \left(y_{gd.sp} - \frac{t_d}{2} \right)^2 + \dots = 1.052 \times 10^{-6}\,mm^4$
$+ n_w \left[\frac{t_w h_w^3}{12} + A_w \left(t_d + \frac{h_w}{2} - y_{gd.sp} \right)^2 \right]$

$n_w = 9$

$\gamma_{sp} = \frac{I_{yd.sp}}{y_{gd.sp}} = 328.143$

$k_\sigma = \frac{n_w A_w}{A_d} = 0.576$

$\alpha_{sp} = \min\left(\frac{b_s}{t_d}, 0.5 \right) = 0.571$ $\sqrt{\gamma_{sp}} = 4.256$

$k_{\sigma.p.sp} = \begin{cases} \frac{2\left[\left(1 + \alpha_{sp}^2 \right)^2 + \gamma_{sp} - 1 \right]}{\alpha_{sp}^2 (\psi_{sp}+1)(1+k_\sigma)} & \text{if } \alpha_{sp} < \sqrt{\gamma_{sp}} = 639.127 \\ \frac{4\left(1 + \sqrt{\gamma_{sp}} \right)}{(\psi_{sp}+1)(1+k_\sigma)} & \text{if } \alpha_{sp} > \sqrt{\gamma_{sp}} \end{cases}$

$\sigma_{cr.sp} := k_{\sigma.p.sp} \sigma_{E.sp} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1.sup.sp} = \frac{3 - \psi_{sp}}{5 - \psi_{sp}} b_{sp} = 0.175\,m$

$b_{2.sup.sp} = \frac{2}{5 - \psi_{sp}} b_{sp} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{c.d.sp} := b_{1.sup.sp} + b_{2.sup.sp} = 0.35\,m$

$y_{gd.c.sp} = \frac{t_d b_{c.d.sp} \frac{t_d}{2} + A_w \left(t_d + \frac{h_w}{2} \right) + A_{td} \left(t_d + \frac{-3}{2} \right)}{t_d b_{c.d.sp} + A_w + A_{td}} = 0.034\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{c.d.sp} = \frac{b_{c.d.sp} t_d^3}{12} + b_{c.d.sp} t_d \left(y_{gd.c.sp} - \frac{t_d}{2} \right)^2 + \frac{t_w h_w^3}{12} + A_w \left(t_d + \frac{h_w}{2} - y_{gd.c.sp} \right)^2$

Combined moment of inertia

$\sigma_{cr.c.sp} = \frac{\pi^2 E I_{c.sp}}{A_{d.c.sp} s_{td}^2} = 1.026 \times 10^3\,MPa$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.sp} = \sigma_{cr.c}$

$\sigma_{cr.c.sp} := \sigma_{cr.c.sp}$

Reduction factor for column-like behaviour

$i_y = \sqrt{\frac{I_{d.c.sp}}{A_{d.c.sp}}} = 0.045\,m$

$e_{sp} = \max\left(y_{gd.c.sp} - t_d - t_d + \frac{h_w}{2} - y_{gd.c.sp}, y_{gd.c.sp} \right) = 0.046\,m$

$\alpha_{e.sp} = 0.49 + \frac{0.09}{\frac{i_y}{e_{sp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{sp} = 0.5\left[1 + \alpha_{e.sp} \left(\lambda_y - 0.2 \right) + \lambda_y^2 \right] = 0.913$ [6.3.1.2]

$\chi_{sp} = \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_y^2} \right)}, 1 \right) = 0.68$

Interpolation

$\xi_{sp} = \min\left(\max\left(\frac{\sigma_{cr.sp.sp}}{\sigma_{cr.c.sp}} - 1.0, 1 \right), 1 \right) = 0$

$\rho_{c.sp} = \begin{cases} \left[\left(\rho_{p.sp} - \chi_{sp} \right) \xi_{sp} \left(2 - \xi_{sp} \right) + \chi_{sp} \right] \text{ if } \frac{\left[\left(\rho_{p.sp} - \chi_{sp} \right) \xi_{sp} \left(2 - \xi_{sp} \right) + \chi_{sp} \right] \gamma_{M1}}{\chi_{sp}} < 1 = 0.68 \\ 1 \text{ if } \frac{\left[\left(\rho_{p.sp} - \chi_{sp} \right) \xi_{sp} \left(2 - \xi_{sp} \right) + \chi_{sp} \right] \gamma_{M1}}{\chi_{sp}} > 1 \end{cases}$

Shear

$\eta_1 = 1.2$ 7.1 (2)

$\chi_w = \begin{cases} \eta_1 \text{ if } \lambda_y < \frac{0.83}{\eta_1} = 1.149 \\ \frac{0.83}{\lambda_y} \text{ if } \frac{0.83}{\eta_1} < \lambda_y < 1.08 \\ \frac{1.37}{(0.7 + \lambda_y)} \text{ if } \lambda_y > 1.08 \end{cases}$

$\frac{\rho_{c.sp} \alpha_{ult}}{\gamma_{M1}} = 0.717$

Side plates check

x-direction

$\sigma_{max} = f_y(t_d)$

$\psi(z) = \left[\frac{\sigma_{min}}{(\gamma_{sp})} \cdot z + \sigma_{max} \right] - 1$

$\psi_{sp.x} = \frac{\sigma\left(h_h - t_d - t_d \right)}{\sigma\left(h_h - t_d \right)} = -0.494$

$\rho_{p.x.sp} = \begin{cases} 1 \text{ if } \lambda_y < 0.673 \\ \frac{\lambda_y - 0.055(4.\psi_{sp})}{\lambda_y^2} \text{ if } \lambda_y > 0.673 \end{cases} = 1.12$

$\frac{\alpha_{ult}}{\gamma_{M1}}$

Plate-like behaviour $\sigma_{x.Ed.sp} = \left(h_h - t_d - y_{gd} \right) = -117.213\,MPa$

$\sigma_{E.sp.x} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) b_s^2} = 36.109\,MPa$ $\psi_{sp} = \frac{\sigma_{x.Ed.sp}}{\sigma_{x.sup}} = -0.494$

$k_{\sigma.sp.x} = \begin{cases} 1 \text{ if } \psi_{sp.x} \ge 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \psi_{sp.x} + 9.78 \psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{sp.x} = -1 \\ 5.98\left(1 - \psi_{sp.x} \right)^2 \text{ if } -1 > \psi_{sp.x} > -3 \\ 95.68 \text{ if } \psi_{sp.x} < -3 \end{cases} = 13.297$

$\sigma_{cr.p.sp.x} = \sigma_{E.sp.x} k_{\sigma.sp.x} = 480.157\,MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} = \frac{\pi^2 E t_s^2}{12\left(1-\nu^2\right) s_{td}^2} = 6.833\,MPa$

$\alpha_{sp} = 0.21$

$\phi_{sp} = 0.5\left[1 + \alpha_{sp} \left(\lambda_y - 0.2 \right) + \lambda_y^2 \right] = 0.816$

$\chi_{sp} = \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_y^2} \right)}, 1 \right) = 0.837$

Interpolation

$\xi_{sp.x} = \min\left(\max\left(\frac{\sigma_{cr.sp.sp.x}}{\sigma_{cr.c.sp.x}} - 1.0, 1 \right), 1 \right) = 1$

$\rho_{c.sp.x} = \begin{cases} \left[\left(\rho_{p.x.sp} - \chi_{sp} \right) \xi_{sp.x} \left(2 - \xi_{sp.x} \right) + \chi_{sp} \right] \text{ if } \frac{\left[\left(\rho_{p.x.sp} - \chi_{sp} \right) \xi_{sp.x} \left(2 - \xi_{sp.x} \right) + \chi_{sp} \right] \gamma_{M1}}{\chi_{sp}} < 1 = 1 \\ 1 \text{ if } \frac{\left[\left(\rho_{p.x.sp} - \chi_{sp} \right) \xi_{sp.x} \left(2 - \xi_{sp.x} \right) + \chi_{sp} \right] \gamma_{M1}}{\chi_{sp}} > 1 \end{cases}$

Z-direction

$\psi_{sp.z} = 1$

$\alpha_{sp.sp} = 0.34$ $\lambda_{sp.sp} = 0.7$

$\phi_{sp.sp} = 0.5\left[1 + \alpha_{sp.sp} \left(\lambda_y - \lambda_{sp.sp} \right) + \lambda_y \right] = 0.865$

$\rho_{p.z.sp} = \begin{cases} 1 \text{ if } \lambda_y < 0.673 \\ \frac{1}{\left(\phi_{sp.sp} + \sqrt{\phi_{sp.sp}^2 - \lambda_y} \right)} \text{ if } \lambda_y > 0.673 \end{cases} = 0.975$

Plate-like behaviour

$k_{\sigma.p.sp.z} = 4$

$\sigma_{E.sp.z} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) s_{td}^2} = 6.833\,MPa$

$\sigma_{cr.p.sp.z} = k_{\sigma.p.sp.z} \sigma_{E.sp.z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{cr.c.sp.z} = \frac{\pi^2 E t_s^2}{12(1-\nu^2) s_d^2} = 36.109\,MPa$

Interpolation

$\xi_{sp.z} = \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1.0, 1 \right), 1 \right) = 0$

$$\rho_{c,up,x} := \begin{cases} \left[\left(\rho_{p,up} - \chi_{up}\right)\left(\kappa_{up,x}\left(2 - \varepsilon_{up,x}\right) - \chi_{up}\right)^2\right] \cdot \alpha \cdot \dfrac{\left[\left(\rho_{p,up} - \chi_{up}\right)\left(\kappa_{up,x}\left(2 - \varepsilon_{up,x}\right) + \chi_{up}\right)^2\right] \cdot \alpha}{\gamma_{M1}} \le 1 \\ 1 \quad \text{if} \quad \dfrac{\left[\left(\rho_{p,up} - \chi_{up}\right)\left(\kappa_{up,x}\left(2 - \varepsilon_{up,x}\right) + \chi_{up}\right)^2\right] \cdot \alpha}{\gamma_{M1}} > 1 \end{cases} = 0.837$$

Top plate stiffeners - check

x-direction

$$\rho_{p,up,y} := \begin{cases} 1 \quad \text{if} \quad \lambda_y < 0.748 \\ \dfrac{\lambda_y - 0.188}{\lambda_y^2} \quad \text{if} \quad \lambda_y > 0.748 \end{cases} = 1$$

$$\psi_{up,x} := \frac{\sigma 1_{up} \cdot \left(z_c - h_{up}\right)}{\sigma 1_{up} \cdot z_c} = 0.76$$

$$\sigma_{E,up,x} := \frac{\pi^2 \cdot E \cdot t_{up}^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_{up}^2} = 2.479 \cdot 10^3 \cdot \text{MPa}$$

$$k_{\sigma,up,x} := \begin{cases} 1 \quad \text{if} \quad \psi_{up,x} \ge 1 \\ \dfrac{8.2}{1.05 + \psi_{up,x}} \quad \text{if} \quad 1 > \psi_{up,x} > 0 \\ 7.81 \quad \text{if} \quad \psi_{up,x} = 0 \\ 7.81 - 6.29 \cdot \psi_{up,x} + 9.78 \cdot \psi_{up,x}^2 \quad \text{if} \quad 0 > \psi_{up,x} > -1 \\ 23.9 \quad \text{if} \quad \psi_{up,x} = -1 \\ 5.98 \cdot \left(1 - \psi_{up,x}\right)^2 \quad \text{if} \quad -1 > \psi_{up,x} > -3 \\ 95.68 \quad \text{if} \quad \psi_{up,x} < -3 \end{cases} = 5.414$$

$$\sigma_{cr,p,up,x} := k_{\sigma,up,x} \cdot \sigma_{E,up,x} = 1.342 \cdot 10^4 \cdot \text{MPa}$$

Column-like behaviour

$$\sigma_{cr,c,up,x} := \frac{\pi^2 \cdot E \cdot t_{up}^2}{12 \cdot \left(1 - \nu^2\right) \cdot a_m^2} = 12.147 \cdot \text{MPa}$$

$$\chi_{up,c} := \chi_{up}$$

Interpolation

$$\xi_{up,x} := \min\left(\left(\frac{\sigma_{cr,c,up,x}}{\sigma_{cr,p,up,x}} - 1.0\right), 1\right) = 1$$

$$\rho_{c,up,x} := \begin{cases} \left[\left(\rho_{p,up,c} - \chi_{up,c}\right)\left(\kappa_{up,x}\left(2 - \varepsilon_{up,x}\right) - \right.\right. \\ \left.\left. - \chi_{up,c}\right)^2\right] \cdot \alpha \cdot \dfrac{\left[\left(\rho_{p,up,c} - \chi_{up,c}\right)\left(\kappa_{up,x}\left(2 - \varepsilon_{up,x}\right) + \chi_{up,c}\right)^2\right] \cdot \alpha}{\gamma_{M1}} \le 1 \\ 1 \quad \text{if} \quad \dfrac{\left[\left(\rho_{p,up,c} - \chi_{up,c}\right)\left(\kappa_{up,x}\left(2 - \varepsilon_{up,x}\right) + \chi_{up,c}\right)^2\right] \cdot \alpha}{\gamma_{M1}} > 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,up}}{\frac{\rho_{c,up,x} \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,up}}{\frac{\chi_w \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right)^2 = 0.989$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,up}}{\frac{\rho_{c,up,x} \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,up}}{\frac{\rho_{c,up,x} \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,up}}{\frac{\rho_{c,up,x} \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right)\left(\frac{\sigma_{x,Ed,up}}{\frac{\rho_{c,up,x} \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right) + 3\left(\frac{\tau_{Ed,up}}{\frac{\chi_w \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right)^2 = 0.491$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,up}}{\frac{\rho_{c,up,x} \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,up}}{\frac{\chi_w \cdot f_y \cdot t_{up}}{\gamma_{M1}}}\right)^2 = 0.451$$

EWM - Changed side plate height

This chapter covers the EWM when changing the side plate height to 950 mm

Geometry

Amount of:

upper stiffeners: $n_{uo} := 9$

Lower stiffeners: $n_{uu} := 6$

Deck subpanels: $n_{s.uo} := n_{uo} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{uo} := \frac{b_d}{n_{s.uo}} = 350 \cdot mm$

Side plate: $h_s := 950mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{uo0} := 20mm$

$s_{uu} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{uo} := 140mm$ $t_{uo} := 16mm$ $a_{uo} := 6mm$

Stiffeners under: $h_{uu} := 120mm$ $t_{uu} := 12mm$ $a_{uu} := 5mm$

$h_{uu.y} := h_{uu} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{h_b}{2} = 1.451 \times 10^{3} \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{h_b}{2} = 1.425 \times 10^{3} \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

$A_d := t_d \cdot b_d$ $A_s := 2 \cdot t_s \cdot h_s$ $A_w := 2 \cdot t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{uo} := n_{uo} \cdot t_{uo} \cdot h_{uo}$ $A_{uu} := n_{uu} \cdot t_{uu} \cdot h_{uu}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{uo} + A_{uu} = 0.135 \cdot m^2$

$A_c := A_{tot} - A_d = 0.055 \cdot m^2$

Change if center of gravity z_{cg}

$z_{cg} := \frac{\begin{pmatrix} A_d \cdot \frac{t_d}{2} + A_{uo}\left(t_d + \frac{h_{uo}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ + A_b\left(t_d + h_s + h_{wy}\right) + A_{uu}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - t_{uu}\right) - \frac{h_{wy}}{2}\right] \end{pmatrix}}{A_{tot}} = 631.079 \cdot mm$

Moment of inertia I_y

$s_{uo0.y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{uo0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.uo} := n_{uo} \cdot \frac{t_{uo} \cdot h_{uo}^3}{12} + n_{uo} \cdot t_{uo} \cdot h_{uo} \left(t_d + \frac{h_{uo}}{2} - z_{tp}\right)^2 = 6.155 \times 10^{-3} \cdot m^4$

$I_{y.st.uu.1} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$I_{y.st.uu} := I_{y.st.uu.1} + 2 \cdot t_{uu} \cdot h_{uu} \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - t_{uu0}\right) - \frac{h_{uu.y}}{2} - z_{tp}\right]^2 \dots = 3.162 \times 10^{-3} \cdot m^4$
$+ 2 \cdot t_{uu} \cdot h_{uu} \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - t_{uu0}\right) - \frac{h_{uu.y}}{2} - z_{tp}\right]^2 \dots$
$+ 2 \cdot t_{uu} \cdot h_{uu} \left[t_d + h_s + \left(h_{wy} - t_{uu0}\right) - \frac{h_{wy}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 \cdot m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 \cdot t_w \cdot h_w \left[h_s - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 0.011 \cdot m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \left(\frac{t_d}{2} - z_{tp}\right)^2 = 0.014 \cdot m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \left(h_s - z_{tp} - \frac{t_b}{2}\right)^2 = 0.011 \cdot m^4$

Side plate:

$I_{y.s} := 2 \cdot \frac{t_s \cdot h_s^3}{12} + 2 t_s \cdot h_s \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 2.201 \times 10^{-3} \cdot m^4$

Total:

$I_{y.tot} := I_{y.st.uo} + I_{y.st.uu} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.047 \cdot m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi \equiv 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi \equiv 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi \equiv -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in compression

$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ 0.57 & \text{if } \psi \equiv 0 \\ 0.85 & \text{if } \psi \equiv -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$k_{\sigma.4.2i}(\psi) := \begin{cases} 0.43 & \text{if } \psi \equiv 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi \equiv 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi \equiv -1 \end{cases}$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2)
Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 960 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.53$

$Condition := \begin{cases} 62 \cdot \varepsilon(t_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$

$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$k_{\sigma.4.1}(\psi_s) = 13.884$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.919$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 0.927$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 328.921 \cdot mm$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 621.079 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 575.651 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 230.26 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 345.391 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 904.572 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 1.382 \times 10^{4} \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 2.171 \times 10^{4} \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12\left(1 - \nu^2\right) h_s^2} = 420.467 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$h_{bv} := t_d + h_{uo} = 10 \cdot mm$

$h_{bv} := t_d + h_{uu} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.775$

$k_{\sigma.4.2d}(\psi_{so}) = 0.519$ Higher stress close to support

$Class4_{so} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon \cdot \left(\frac{1}{3}\right) \cdot \sqrt{k_{\sigma.4.2d}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p\left(b_{\sigma.4.2d}(\psi_{so}), b_{so}, t_{so}\right) = 0.526$

$\rho_{d.so} := \rho_{4.2}\left(\lambda_{p.so}, \psi_d\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so} \cdot b_{so} = 140\,mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot b_{so.eff} = 2.016 \times 10^{-4}\,m^2$

Deck

$\frac{b_{sd}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{sd} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{sd}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3}\,m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$n_{sw} := n_{so} \qquad$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p\left(k_{\sigma.d} \cdot s_{c.so}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461\,mm$

Areas:

$A_{sw} := n_{so} \cdot s_{so} \cdot t_d + n_{so} \cdot s_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.31 \times 10^{-4}\,m^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot s_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.053 \times 10^{-4}\,m^2$

More than three longitudinal stiffeners --> $k_{\sigma.d}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^{-2}\,mm^4$

$y_{tp.sl} := \frac{n_{sw} \cdot t_{so} \cdot b_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot b_{so} + b_d \cdot t_d} = 32.411\,mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \dots + \left[\frac{n_{sw} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{-6}\,mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \qquad \alpha := \frac{a}{h_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot b_{so} \cdot t_{so} = 0.02\,m^2$ EC1993-1-5A.1 (2)

$A_p := b_d \cdot t_d = 0.035\,m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952 \cdot \left[\frac{1}{\alpha \cdot \sqrt{1 - \psi}}\right] \text{ if } \alpha \le \frac{4}{\sqrt{1 - \psi}} = 639.127$

$\lambda_{p.c} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$ herwise

$\rho := \rho_{4.2}\left(\lambda_p, \psi\right) = 1$ orthotropic plate

Plate buckling behaviour

$E = 2.1 \times 10^5\,MPa \qquad \nu = 0.3 \qquad \sigma_{cr.p} := \frac{E \cdot t_d^2}{21 \cdot \left(1 - \nu^2\right) \cdot b_d^2} = 236.4\,MPa$ EC1993-1-5 eq.A.1

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, \psi\right) \cdot A_{c.eff.loc} + n_{so.eff} \cdot t_d = 5.375 \times 10^{-4}\,m^2$

Bucking strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^4\,kN$

Column like behaviour

$y_{tp.sl.c} := \frac{n_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + n_{so} \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot h_{so} + n_{so} \cdot t_d} = 34.268\,mm$

$I_{sl.c} := \left[\frac{n_{so} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \dots + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{-7}\,mm^4$

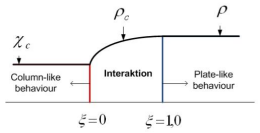
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot b_{so} + s_{so} \cdot t_d = 5.74 \times 10^{-3}\,mm^2$

$A_{sl.eff} := t_{so} \cdot b_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{-3}\,mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.026 \times 10^3\,MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sd})}{\sigma_{cr.sl}}} = 0.573$

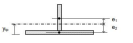
calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\,mm$

$\bar{e}_c := \max\left(\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right) = 47.589\,mm$



$\alpha_e := \alpha + \frac{0.09}{\frac{i}{\bar{e}}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot \left(\lambda_c - 0.2\right) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1\right), 1\right), 0\right) = 0$

ξ is zero --> column like behviour

Interaktion mellan de två olika beteendena(column/plate buckling):

$\rho_c := \left(\rho - \chi_c\right) \cdot \xi \cdot \left(2 - \xi\right) + \chi_c = 0.792$

Checks

Mean stress not

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(t_d + \frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.644 \times 10^{-4}\,kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -334.862\,MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sd})}{\gamma_M} = 281.032\,MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{sd})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{so} \cdot s_{so.eff} \cdot t_d = 0.032\,m^2$

$A_{tot.eff} := A_{sw.eff} + A_{d.eff} + A_w + A_{fo} + A_b + A_{c.eff} = 0.131\,m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{sw.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ &\left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right)\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{fo} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{so.ft}\right) - \frac{h_{so.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{tb} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 647.805\,mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 2.157 \times 10^{-3}\,m^4$

$I_{y.d.eff} := \frac{n_{so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{so} \cdot s_{so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 0.013\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.049\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{d.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^{-4}\,m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_d) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.887 \times 10^{-3}\,mm^2$

$A_{sd.eff.neg} := A_{fo}$ Bottom flange in tension

A_{fo} Tension -> No reduction

The effective area in compression is calculated according to 4.5.1(4.5). If the $A_{c.eff.loc.mod}$ from eq.A.2.1(4) which was checked before is not fulfilled, further reduction is ...ided and the stress in the plate is in class 4.

$A_{so} := \begin{cases} \text{Stiffener class 4 --> no reduction} \\ \frac{\rho_c \cdot f_y(t_d) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M \cdot n_{so} \cdot s_{so.eff} \cdot t_d} & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{sl.fac} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 432.173\,cm^2$

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{sw.eff} + A_w + A_{fo} + A_b = 1.217 \times 10^{-3}\,cm^2$

A_{so} Lower stiffener in tension -> no reduction

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d \end{cases} = 7.916\,mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot b_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) \dots \\ &+ n_{so} \cdot t_{d.eff} \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) \dots \\ &+ t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ &+ A_{fo} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{so.ft}\right) - \frac{h_{so.y}}{2}\right] + A_b \cdot \left(h_{tb} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 648.922\,mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 0.013\,m^4$

$\alpha_{so.fin} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{so.fi} = 7.206\,mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so} \cdot \frac{s_{so} \cdot s_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot s_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 6.558 \times 10^{-3}\,m^4$

$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots + 2 \cdot s_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3} \cdot \left(h_{wy} - s_{suby}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot s_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3} \cdot \left(h_{wy} - s_{suby}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot s_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{suby}\right) - \frac{h_{so.y}}{2} - z_{tp.eff.neg}\right]^2 = 2.982 \times 10^{-3}\,m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[h_{tb} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 0.01\,m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{tb} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 0.01\,m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\right] = 2.165 \times 10^{-3}\,m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 4.556 \times 10^{-10}\,mm^4$

Load

Load capacity:

$q_{.M} := 175\,\frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is $q_{.M}$ when checking the moment capacity.

$Q_{.M} := q_{.M} \cdot b_d = 612.5\,\frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 2.481 \times 10^4\ kN \cdot m$

$q_V := 154 \frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V \cdot b_d = 539 \frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.851 \times 10^3\ kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{sp.eff.neg}} = 0.07\ m^2$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 2.492 \times 10^4\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.995$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 S.5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15d on each side of the stiffener is used when calculating the moment of inertia.

$h_{web} := 2 \cdot 15 \cdot t(t_w) \cdot t_w = 0.293\ m$

$\alpha_w := asin\left(\frac{h_{wz}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{sub} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ss}}{2} - \frac{sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{ss}}{2} = 0.503\ m$

$a_{st.z.2} := a_{st.z.1} + s_{ss} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\ m$

$a_{st.z.3} := a_{st.z.2} + s_{ss} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\ m$

Moment of inrtia stiffeners z-direction

$I_{st.z} := 2 \cdot t_{ss} \cdot h_{ss} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\ m^4$

Distance from z-axis to middle of the web area which is 15d each side of the stiffeners:

$\bar{a}_{web.1} := \frac{b_b}{2} + s_{sub} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ss}}{2} = 0.525\ m$

$\bar{a}_{web.2} := a_{st.z.1} + s_{ss} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\ m$

$\bar{a}_{web.3} := a_{st.z.2} + s_{ss} \cdot cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\ m$

Moment of inertia of 15d of the web z-direction

$I_{stw.z} := 2 \cdot b_{web} \cdot t_w \cdot \left[\left(\bar{a}_{st.z.1}\right)^2 + \left(\bar{a}_{st.z.2}\right)^2 + \left(\bar{a}_{st.z.3}\right)^2\right] = 0.021\ m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\ m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{otherwise} \end{cases} = 6.158$

$k_{\tau 2} := \max\left[9 \cdot \left(\frac{h_w}{a}\right) \cdot \sqrt[4]{\left(\frac{I_{st.z.tot}}{\left(\min(t_f, t_w)\right)^3 \cdot h_w}\right)^3}, \frac{2.1}{\min(t_f, t_w)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^3$

$37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_{\tau 1}}$ eq.5.30

SS-EN 1993-1-5 A.3 (1) och (2) ger:

Shear buckling coefficient, web with more than two longitudinal stiffeners

$k_\tau := \begin{cases} 5.34 + 4 \cdot \left(\frac{h_{web}}{a}\right)^2 + k_{\tau 1} & \text{if } \alpha \geq 3 \\ 6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min(t_f, t_w)^3 \cdot h_{web}} + \frac{2.2}{\alpha^2} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min(t_f, t_w)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.008 \times 10^3$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\gamma_M = 1.2$ EC1993-1-3 S.ANM.2 S.1

$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_f, t_w)} \leq 31 \cdot \frac{\varepsilon(t_f)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases}$ = "Reduce"

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := max\left(\frac{h_w}{37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_\tau}}, \lambda_{w1}\right) = 0.998$ SS-EN 1993-1-5 S.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 0.831$ SS-EN 1993-1-5 S.3 Tabell 5.1, stiff end.

Kapacity of the web

$\gamma_{M0} = 1.00$ $\gamma_{M1} := 1.2$ Safety class 3

$\gamma_{M1} = 1.00$ SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 2.428 \times 10^3\ kN$ SS-EN 1993-1-3 S.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.851 \times 10^3\ kN$

$\frac{V_{Ed}}{2 \cdot V_{bw.Rd}} = 0.999$ (contribution from two webs)

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} \cdot 1$

Kapacity of the flanges

SS-EN 19931-5 S.5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$z_{tf.eff.neg} := z_{sp.eff.neg} - \frac{t_d}{2} = 643.922\ mm$

$z_{af.eff.neg} := h_b - \frac{t_b}{2} - z_{sp.eff.neg} = 793.878\ mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \cdot \left|z_{tf.eff.neg}\right| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{af.eff.neg} \cdot \left|z_{af.eff.neg}\right| \cdot f_y(t_b)}{\gamma_{M0}} = 1.186 \times 10^4\ kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{af} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_b \cdot \left(h_b + h_{wz}\right)^2 \cdot f_y(t_b)}\right] = 0.545\ m$

$c_{tf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_b \cdot \left(h_b + h_{wz}\right)^2 \cdot f_y(t_b)}\right] = 0.533\ m$

$V_{bf.Rd.cored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{af} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{af} \cdot \gamma_{M1} \cdot \gamma_n} = 331.844\ kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.cored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\ kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \leq \frac{\eta \cdot f_y(t_w) \cdot h_{web} \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} \cdot 1$ EC1993-1-5 Ch.5.2(1)

$V_{shear} := V_{bw.Rd} + V_{bf.Rd} = 2.428 \times 10^3\ kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \leq 1 \cdot 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 \cdot V_{b.Rd}} = 0.999$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.023\ m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{ss} \cdot s_{so.eff} \cdot t_{d.eff} + n_{ss} \cdot s_{so} \cdot t_{d.eff} + A_b = 432.579\ cm^2$

$z_{sp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{ss} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \cdots + n_{ss} \cdot s_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_b - \frac{t_b}{2}\right)}{A_{eff.f.tot}} = 536.227\ mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{sp.f.eff.neg}} = 0.044\ m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.55 \times 10^4\ kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 2.492 \times 10^4\ kN \cdot m$

Moment capacity check when $q_M = 175 \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.995$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 154 \frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 \cdot V_{bw.Rd}} = 0.999$

C. Box girder

The Reduced stress method

-Changed side plate height to 950 mm

Material

$f_{y0} := \begin{cases} 355\,MPa & \text{if } t \le 40mm \\ 335\,MPa & \text{if } t > 40mm \end{cases}$ $\gamma_{M1} = 1$

$E = 210GPa$

$\nu = 0.3$ $G = \frac{E}{2\cdot(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$t_b = 16mm$ $b_b = 1000mm$ $A_b = b_b \cdot t_b = 0.016m^2$

Top plate

$t_d = 10mm$ $b_d = 3500mm$ $A_d = b_d \cdot t_d = 0.035m^2$

Side plates

$t_s = 12mm$ $h_s = 950mm$ $A_s = h_s \cdot t_s = 0.011m^2$

Webs

$t_w = 12mm$ $h_w = 1340mm$ $A_w = h_w \cdot t_w = 0.016m^2$

$b_{w.s} = 1250mm$ $b_{w.p} = 482.8mm$

Top stiffeners

$t_{st} = 16mm$ $h_{st} = 140mm$ $A_{st} = h_{st} \cdot t_{st} = 2.24 \cdot 10^{-3}m^2$

$s_{st} = 350mm$ $n_{st} = 9$

Web stiffeners

$t_{st} = 12mm$ $h_{st} = 120mm$ $A_{st} = h_{st} \cdot t_{st} = 1.44 \cdot 10^{-3}m^2$

$s_{st} = 440mm$ $n_{st} = 3$

Whole bridge

$t_{whbdge} = 18m$ $h_{tt} = \frac{t_d}{2} + h_w + t_b + t_d = 1.451m$ $t_{fp} = 2m$

$I_{yy} = 0.046844m^4$ From grasshopper

$V_{fp} = 0.819948m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q_1 = 126\frac{kN}{m^2}$

$Q = q_1 \cdot t_{fp} = 420\frac{kN}{m}$

$M_{Ed} = \frac{Q \cdot t_{whbdge}^2}{8} = 1.701 \cdot 10^4\,kN \cdot m$

$V_{Ed} = \frac{Q \cdot t_{whbdge}}{2} = 3.78 \cdot 10^3\,kN$

Top plate

$\sigma_{x.Ed.tp} = \frac{M_{Ed}}{I_{yy}} \cdot \left(h_w - \frac{t_d}{2} - y_{tp}\right) = 227.259\,MPa$ $\sigma_{z.Ed.tp} = 0$

$I_{yy} = \frac{t_d \cdot b_d}{n_{st} \cdot 2} \cdot \left(h_w - \frac{t_d}{2} - y_{tp}\right)^2 = 1.991 \cdot 10^{-3}\,m^3$

$\tau_{Ed.tp} = \frac{V_{Ed} \cdot S_{yy}}{I_{yy} \cdot \min(t_w, t_{st})} = 13.391\,MPa$

Side plates

$\sigma_{x.Ed.sp} = \frac{M_{Ed}}{I_{yy}} \cdot (h_w - t_d - y_{tp}) = 225.444\,MPa$

$\sigma_{z.Ed.sp} = \frac{Q}{2 \cdot t_s} = 17.5\,MPa$

$S_{yy} = (h_w - t_d - y_{tp}) \cdot t_s \cdot \frac{(h_w - t_d - y_{tp})}{2} + \frac{t_d \cdot b_d}{2} \cdot \left(h_w - \frac{t_d}{2} - y_{tp}\right) = 0.019\,m^3$
$+ \frac{h_{st} \cdot t_{st}}{2} \cdot n_{st} \cdot \left(h_w - t_d - \frac{h_{st}}{2} - y_{tp}\right)$

$S_{yy.s} = \frac{\left(h_w - \frac{t_{st} \cdot h_{st}}{2} - 2\frac{t_d \cdot b_d}{2}\right)}{n_{st} + 2} \cdot \left(h_w - t_d - \frac{h_{st}}{4} - y_{tp}\right) = 2.618 \cdot 10^{-3}\,m^3$
$\tau_{td.sp} = \frac{V_{Ed} \cdot S_{yy.s}}{I_{yy} \cdot t_s}$

Top plate stiffeners

$\tau_{Ed.tpst} = \frac{V_{Ed} \cdot S_{yy}}{I_{yy} \cdot t_{st}} = 13.202\,MPa$ $\frac{\sigma_{x.Ed.tp}}{\gamma_{M1}} = 225.444\,MPa$

Governing slenderness from FE-software

$\sigma_{y.UNIT} = 2.61\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

$\sigma_{cr} = \sigma_{y.UNIT} \cdot q_1 = 313.2\,MPa$

$\alpha_{ult} = \frac{f_y(t_d)}{\sigma_{cr}} = 1.133$

$\sigma_{cr.UNIT} = 223.66\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} = \frac{\sigma_{cr.UNIT}}{q_1} = 1.864$

$\lambda_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.78$ Slenderness for all parts

Top plate check

$\psi_{tp} = 1$

$\rho_{tp.tp} = \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.921$

Plate-like behaviour [A.1]

$\sigma_{E.tp} = \frac{\pi^2 \cdot E \cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\,MPa$

$I_{yy.tp} = \frac{b_d \cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205 \cdot 10^{-7}\,m^4$

$y_{tp.sp} = \frac{A_d \cdot \frac{t_d}{2} + h_w \cdot A_w \cdot \left(t_d + \frac{h_w}{2}\right)}{A_d + n_w \cdot A_w} = 32.411\,mm$

$I_{yy.tp} = \frac{b_d \cdot t_d^3}{12} + A_d \cdot \left(y_{tp.sp} - \frac{t_d}{2}\right)^2 + \dots = 1.052 \cdot 10^{-8}\,mm^4$
$+ n_w \cdot \left[\frac{t_w \cdot h_w^3}{12} + A_w \cdot \left(t_d + \frac{h_w}{2} - y_{tp.sp}\right)^2\right]$

$n_w = 9$

$\gamma_{tp} = \frac{t_{fp.sp}}{y_{tp.sp}} = 328.143$

$k_\sigma = \frac{n_{st} \cdot A_{st}}{A_d} = 0.576$

$\alpha_{tp} = \min\left(\frac{2a}{b_d}, 0.5\right) = 0.571$ $\sqrt{\gamma_{tp}} = 4.256$

$k_{\sigma.p.tp} = \begin{cases} \frac{2\cdot\left[\left(1+\alpha_{tp}^2\right)^2\cdot\gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}-1)\cdot(1+k_\sigma)} & \text{if } \alpha_{tp} \cdot \sqrt{\gamma_{tp}} = 639.127 \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}-1)\cdot(1+k_\sigma)} & \text{if } \alpha_{tp} \cdot \sqrt{\gamma_{tp}} < 639.127 \end{cases}$

$\sigma_{cr.tp} = k_{\sigma.p.tp} \cdot \sigma_{E.tp} = 990.256\,MPa$

Column-like behaviour [4.5.3]

$b_{1.tp.tp} = \frac{3-\psi_{tp}}{5-\psi_{tp}} \cdot t_{fp} = 0.175m$

$b_{2.tp.tp} = \frac{2}{5-\psi_{tp}} \cdot t_{fp} = 0.175m$

Parts of the top plate that contributes to the stiffener

$b_{ef.1.tp} = b_{1.tp.tp} + b_{2.tp.tp} = 0.35m$

$y_{tp.ef.1.tp} = \frac{t_d \cdot b_{ef.1.tp} \cdot \frac{t_d}{2} + A_{st} \cdot \left(t_d + \frac{h_{st}}{2}\right)}{t_d \cdot b_{ef.1.tp} + A_{st}} = 0.034m$ combined center of gravity for stiffener and contributing top plate.

$I_{ef.1.tp} = \frac{b_{ef.1.tp} \cdot t_d^3}{12} + b_{ef.1.tp} \cdot t_d \cdot \left(y_{tp.ef.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{st} \cdot h_{st}^3}{12} + A_{st} \cdot \left(t_d + \frac{h_{st}}{2} - y_{tp.ef.1.tp}\right)^2$
Combined moment of inertia

$\sigma_{cr.ef.tp} = \frac{\pi^2 \cdot E \cdot I_{yy.tp}}{A_{ef.1.tp} \cdot b_d^2} = 1.026 \cdot 10^3\,MPa$ critical stress for kolumnike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.st} = \sigma_{cr.ef}$

$\sigma_{cr.c.tp} = \sigma_{cr.ef.tp}$

Reduction factor for column-like behaviour

$i_{tp} = \sqrt{\frac{I_{yy.tp}}{A_{ef.1.tp}}} = 0.045m$

$e_{tp} = \max\left(\left|y_{tp.ef.1.tp} - t_d - y_{tp} + \frac{h_{st}}{2}\right|, \left|-y_{tp.ef.1.tp}\right|\right) = 0.046m$

$\alpha_{e.tp} = 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} = 0.5\cdot\left[1 + \alpha_{e.tp}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.973$ [6.3.1.2]

$\chi_{tp} = \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}}, 1\right) = 0.643$

Interpolation

$\xi_{tp} = \max\left(\min\left(\frac{\sigma_{cr.c.tp}}{\sigma_{cr.ef.tp}} - 1.0\right), 1\right) = 0$

$\rho_{c.tp} = \begin{cases} \left[\left(\rho_{tp.tp} - \chi_{tp}\right)\cdot\xi_{tp}\cdot\left(2-\xi_{tp}\right) + \chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{tp.tp} - \chi_{tp}\right)\cdot\xi_{tp}\cdot\left(2-\xi_{tp}\right) + \chi_{tp}\right]\cdot n_{st}}{\gamma_{M1}} \le 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{tp.tp} - \chi_{tp}\right)\cdot\xi_{tp}\cdot\left(2-\xi_{tp}\right) + \chi_{tp}\right]\cdot n_{st}}{\gamma_{M1}} > 1 \end{cases} = 0.643$

Shear

$\eta_1 = 1.2$ 7.1 (2)

$k_\tau = \begin{cases} \eta_1 & \text{if } \lambda_p < \frac{0.83}{\eta_1} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta_1} \le \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.064$

$\frac{\sigma_{cr.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.729$

Side plates check

x-direction

$\sigma_{max} = f_y(t_d)$

$\sigma_{CO} = \left|\frac{\sigma_{min}}{f(t_d)}\right| = -1$

$\psi_{sp.x} = \frac{\sigma\left(h_w - t_d - y_{tp}\right)}{\sigma\left(h_w - t_d\right)} = -0.53$

$\sigma_{x.sp} = \frac{M_{Ed}}{I_{yy}} \cdot (h_w - t_d - y_{tp}) = 225.444\,MPa$

Plate-like behaviour $\frac{M_{Ed}}{I_{yy}} \cdot y_{tp} = 52\,MPa$

$\sigma_{E.sp.x} = \frac{\pi^2 \cdot E \cdot t_s^2}{12\cdot(1-\nu^2)\cdot h_w^2} = 30.284\,MPa$ $\psi_{sp.x} = \frac{\sigma_{x.Ed}}{\sigma_{x.sup}} = -0.53$

$\psi_{sp.x} = \max(3 + \psi_{sp.x}, 0) = 2.47$

$\rho_{p.sp.x} = \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(\psi_{sp.x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1.059$

$k_{\sigma.sp.x} = \begin{cases} 1 & \text{if } \psi_{sp.x} \ge 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29\cdot\psi_{sp.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot(1-\psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} > -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 13.893$

$\sigma_{cr.p.sp.x} = \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 420.749\,MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} = \frac{\pi^2 \cdot E \cdot t_s^2}{12\cdot(1-\nu^2)\cdot h_w^2} = 6.833\,MPa$

$\alpha_{sp} = 0.21$

$\phi_{sp} = 0.5\cdot\left[1 + \alpha_{sp}\cdot\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.865$

$\chi_{sp} = \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}}, 1\right) = 0.807$

Interpolation

$\xi_{sp.x} = \max\left(\min\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1.0\right), 1\right) = 1$

$\rho_{c.sp.x} = \begin{cases} \left[\left(\rho_{tp.sp} - \chi_{sp}\right)\cdot\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{tp.sp} - \chi_{sp}\right)\cdot\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right) + \chi_{sp}\right]\cdot n_{st}}{\gamma_{M1}} \le 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{tp.sp} - \chi_{sp}\right)\cdot\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right) + \chi_{sp}\right]\cdot n_{st}}{\gamma_{M1}} > 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} = 1$

$\alpha_{sp.sp} = 0.34$ $\chi_{tp.sp} = 0.7$

$\phi_{sp.sp} = 0.5\cdot\left[1 + \alpha_{sp.sp}\cdot\left(\lambda_p - \chi_{tp.sp}\right) + \lambda_p^2\right] = 0.903$

$\rho_{p.sp.sp} = \begin{cases} 1 & \text{if } \lambda_p < 0.673 \\ \frac{1}{\phi_{sp.sp} + \sqrt{\phi_{sp.sp}^2 - \lambda_p^2}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.914$

Plate-like behaviour

$\psi_{sp.sp.z} = 4$

$\sigma_{E.sp.z} = \frac{\pi^2 \cdot E \cdot t_s^2}{12\cdot(1-\nu^2)\cdot h_w^2} = 6.833\,MPa$

$\sigma_{cr.p.sp.z} = k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 27.331\,MPa$

Column-like behaviour

$\sigma_{cr.c.sp.z} = \frac{\pi^2 \cdot E \cdot t_s^2}{12\cdot(1-\nu^2)\cdot h_w^2} = 30.284\,MPa$

Interpolation

$\xi_{sp.z} = \max\left(\min\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1.0\right), 1\right) = 0$

$\rho_{c.sp.z} = \begin{cases} \left[\left(\rho_{tp.sp} - \chi_{sp}\right)\cdot\xi_{sp.z}\cdot\left(2-\xi_{sp.z}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{tp.sp} - \chi_{sp}\right)\cdot\xi_{sp.z}\cdot\left(2-\xi_{sp.z}\right) + \chi_{sp}\right]\cdot n_{st}}{\gamma_{M1}} \le 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{tp.sp} - \chi_{sp}\right)\cdot\xi_{sp.z}\cdot\left(2-\xi_{sp.z}\right) + \chi_{sp}\right]\cdot n_{st}}{\gamma_{M1}} > 1 \end{cases} = 0.807$

CCC

Top plate stiffeners - check

x-direction

$$\rho_{p,c,sp,x} = \begin{cases} 1 & \text{if } \lambda_{p,x} < 0.748 \\ \frac{\lambda_{p,x} - 0.188}{\lambda_{p,x}^2} & \text{if } \lambda_{p,x} > 0.748 \end{cases} = 0.973$$

$$\psi_{sp,c,x} = \frac{\sigma_2\left(b_{sp,x} - t_p - h_{sw}\right)}{\sigma_1\left(b_{sp,x} - t_p\right)} = 0.775$$

$$\sigma_{E,sp,c,x} = \frac{\pi^2 E_s t_{sp}^2}{12\left(1 - \nu^2\right) b_{sp,x}^2} = 2.479 \cdot 10^3 \text{ MPa}$$

$$k_{\sigma,sp,c,x} = \begin{cases} 1 & \text{if } \psi_{sp,c,x} \geq 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp,c,x}\right)} & \text{if } 1 > \psi_{sp,c,x} > 0 \\ 7.81 & \text{if } \psi_{sp,c,x} = 0 \\ 7.81 - 6.29 \psi_{sp,c,x} + 9.78 \psi_{sp,c,x}^2 & \text{if } 0 > \psi_{sp,c,x} > -1 \\ 23.9 & \text{if } \psi_{sp,c,x} = -1 \\ 5.98\left(1 - \psi_{sp,c,x}\right)^2 & \text{if } -1 > \psi_{sp,c,x} > -3 \\ 95.68 & \text{if } \psi_{sp,c,x} < -3 \end{cases} = 5.687$$

$$\sigma_{cr,sp,c,x} = k_{\sigma,sp,c,x} \sigma_{E,sp,c,x} = 1.41 \cdot 10^4 \text{ MPa}$$

Column-like behaviour

$$\sigma_{cr,c,sp,c,x} = \frac{\pi^2 E_s t_{sp}^2}{12\left(1 - \nu^2\right) b_{sp}^2} = 12.147 \text{ MPa}$$

$$b_{sp,c} = b_{sp}$$

Interpolation

$$\xi_{sp,c,x} = \min\left(\max\left(\frac{\sigma_{cr,sp,c,x}}{\sigma_{cr,c,sp,c,x}} - 1.0\right), 1\right) = 1$$

$$\rho_{c,sp,c,x} = \begin{cases} \left[\left(\rho_{sp,c,sp} - \chi_{sp}\right) h_{sp,c,x} \left(2 - \xi_{sp,c,x}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{sp,c,sp} - \chi_{sp}\right) h_{sp,c,x} \left(2 - \xi_{sp,c,x}\right) + \chi_{sp}\right] \sigma_{sd}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{sp,c,sp} - \chi_{sp}\right) h_{sp,c,x} \left(2 - \xi_{sp,c,x}\right) + \chi_{sp}\right] \sigma_{sd}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} f_y\left[t_p\right]}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w f_y\left[t_p\right]}{\gamma_{M1}}}\right)^2 = 0.594$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} f_y\left[t_{sw}\right]}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} f_y\left[t_{sw}\right]}{\gamma_{M1}}}\right)^2 \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} f_y\left[t_{sw}\right]}{\gamma_{M1}}}\right) = 0.452$$
$$+ 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w f_y\left[t_{sw}\right]}{\gamma_{M1}}}\right)^2$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} f_y\left[t_s\right]}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_w f_y\left[t_s\right]}{\gamma_{M1}}}\right)^2 = 0.407$$

EWM - Changed side plate height

This chapter covers the EWM when changing the side plate height to 1030 mm.

Geometry

Amount of:

upper stiffeners: $n_{su} := 9$

Lower stiffeners: $n_{sl} := 6$

Deck subpanels: $n_{s.su} := n_{su} + 1 = 10$

Deck: $h_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{su} := \frac{b_d}{n_{s.su}} = 350 \cdot mm$

Side plate: $h_s := 1030mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 1.531 \times 10^{-3} \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 1.505 \times 10^{-3} \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{-\sigma_{max}}{z_{tp}} z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$

Buckling factors from table 4.1 SS-EN 1993-1-5 Internal parts in compression

Area

$A_d := t_d \cdot h_d$ $A_s := 2 t_s \cdot h_s$ $A_w := 2 t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$ $A_{sl} := n_{sl} \cdot t_{sl} \cdot h_{sl}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{su} + A_{sl} = 0.137 \cdot m^2$

$A_c := A_{tot} + A_d = 0.055 \cdot m^2$ 3mm EC1993-1-1 tab. 3.1

(355MPa) otherwise

Change of center of gravity, z_{tp}

$\Delta(i) := \sqrt{\frac{355MPa}{f_y(i)}}$

$z_{tp} := \frac{\begin{pmatrix} A_d \frac{t_d}{2} + A_{su}\left(t_d + \frac{h_{su}}{2}\right) + A_s\left(t_d + \frac{h_s}{2}\right) + A_w\left(t_d + h_s + \frac{h_{wx}}{2}\right) \dots \\ + A_b\left(t_d + h_s + h_{wy}\right) + A_{sl}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{sl0}\right) - \frac{h_{sl.y}}{2}\right] \end{pmatrix}}{A_{tot}} = 669.507 \cdot mm$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners overlunder:

$I_{y.st.so} := n_{so} \frac{h_{so} \cdot a_{so}^3}{12} + n_{so} t_{so} h_{so} \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 7.039 \times 10^{-3} \cdot m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} \cdot m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2 n_{su} h_{su} \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 3.6 \times 10^{-3} \cdot m^4$

$+ 2 n_{su} h_{su} \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$

$+ 2 n_{su} h_{su} \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2 h_w t_w \left[h_s - \frac{h_b}{2} - \frac{1}{2}\left(h_{wy}\right) - z_{tp}\right]^2 = 0.013 \cdot m^4$

Deck:

$I_{y.d} := \frac{b_d t_d^3}{12} + t_d b_d \left(\frac{t_d}{2} - z_{tp}\right)^2 = 0.015 \cdot m^4$

Bottom flange:

$I_{y.b} := \frac{b_b h_b^3}{12} + b_b h_b \left(h_s - z_{tp} - \frac{t_b}{2}\right)^2 = 0.012 \cdot m^4$

Side plate:

$I_{y.s} := 2 \frac{t_s h_s^3}{12} + 2 t_s h_s \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 2.702 \times 10^{-3} \cdot m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.053 \cdot m^4$

$k_{\sigma.4.2s}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$

Buckling factors from table 4.2 SS-EN 1993-1-5 upper part Elements in compression with free edge. Lower stress close to support.

$k_{\sigma.4.2d}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$

Buckling factors from table 4.2 SS-EN 1993-1-5 lower part Elements in compression free edge. Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(i) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \cdot \max(3 + \psi, 0)}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.2) Internal parts

$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$

SS-EN 1993-1-5 (4.3) Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 1.04 \times 10^{-3} \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.562$

$\text{Condition} := \begin{cases} 62 \varepsilon(i_s)\left(1 - \psi_s\right)\sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(i_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$

$\text{Class4} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > \text{Condition} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$k_{\sigma.4.1}(\psi_s) = 14.43$

$s_{c.so} := s_{so} - s_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.978$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 0.882$ Internal cross section parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 370.493 \cdot mm$ Height, tension part

$h_{s.c} := h_s - h_{s.t} = 659.507 \cdot mm$ Height, compression part

$h_{s.eff.c} := \rho_s(h_{s.c}) = 581.938 \cdot mm$

$h_{s.eff.c1} := 0.4 h_{s.eff.c} = 232.775 \cdot mm$

$h_{s.eff.c2} := 0.6 h_{s.eff.c} = 349.163 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 952.432 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 1.397 \times 10^{-3} \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 2.286 \times 10^{-4} \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \frac{E \cdot \pi^2 \cdot s_s^2}{12\left(1 - \nu^2\right) h_s^2} = 371.75 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$\Delta t_w := t_d = 10 \cdot mm$

$d_{sw} := t_d + h_{so} = 150\ mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.788$

$k_{\sigma.4.2ll}(\psi_{so}) = 0.513$ Higher stress close to support

$Class4_{sl} := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \epsilon \cdot (\zeta_{so}) \cdot \sqrt{k_{\sigma.4.2ll}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2ll}(\psi_{so}), b_{so}, t_{so}) = 0.529$

$\rho_{d.so} := \rho_{4.2}(\lambda_{p.so}, \psi_{so}) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$b_{so.eff} := \rho_{d.so} \cdot b_{so} = 140\ mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot b_{so.eff} = 2.016 \times 10^{-6}\ m^2$

Deck

$\frac{b_{sw}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_{fl} := \begin{cases} \text{"Class4"} & \text{if } \frac{b_{sw}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_{sl} := \frac{-t_d}{2} = -5 \times 10^{-3}\ m$ $\psi_{sl} := \psi_{neg}(z_{sl}, z_{sl}) = 1$

$b_{sw} := b_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.sl} := k_{\sigma.4.1}(\psi_{sl}) = 4$

$\lambda_{p.sl} := \lambda_p(b_{\sigma.sl} \cdot t_{so}, t_{sl}) = 0.723$

$\rho_{sl} := \rho_{4.2}(\lambda_{p.sl}, \psi_{sl}) = 0.962$ Internal cross-sectional parts

$t_{so.eff} := \rho_{sl} \cdot t_{so} = 321.461\ mm$

Areas:

$A_{sw} := n_{so} \cdot t_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.31 \times 10^{-4}\ m^2$

$A_{c.eff.loc} := n_{so} \cdot t_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.053 \times 10^{-4}\ m^2$

More than three longitudinal stiffeners -> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{-5}\ m^4$

$y_{sp.sl} := \frac{n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} \cdot t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411\ mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl}\right)^2\right] \dots + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl}\right)^2\right] = 1.052 \times 10^{-8}\ m^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \quad \alpha := \frac{s}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02\ m^2$ EC1993-1-5A.1 (2)

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := \frac{E \cdot \pi^2 \cdot I_p^2}{12 \cdot (1 - \nu^2) \cdot b_d^2 \cdot t_d^2} = 990.256\ MPa$ EC1993-1-5 eqA.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952 \qquad \sigma_{cr.p} := \begin{cases} \left(\frac{\alpha}{1} + \frac{1}{\alpha}\right)^2 \cdot \sigma_{cr.p} & \text{if } \alpha \le \sqrt{\gamma} \\ \left(1 + \sqrt{\gamma}\right)^2 \cdot \sigma_{cr.p} & \text{otherwise} \end{cases} = 639.127$

$\bar{\lambda}_{sw} := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}(\bar{\lambda}_p, \psi) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\bar{\lambda}_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 5.375 \times 10^{-4}\ m^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.908 \times 10^{3}\ kN$

Column like behaviour

$y_{sp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268\ mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^3}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{sp.sl.c}\right)^2\right] \dots + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl.c}\right)^2\right] = 1.137 \times 10^{-7}\ m^4$

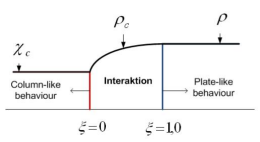
Bucking resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{-3}\ m^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{-3}\ m^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot s^2} = 1.026 \times 10^{3}\ MPa$

Slenderness ratio

$\bar{\lambda}_c := \sqrt{\frac{\beta_c \cdot f_y(t_{sw})}{\sigma_{cr.sl}}} = 0.573$

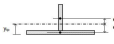
calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_c := 0.49$ Stiffeners open cross-sections

$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\ mm$

$e_c := \max\left[\left(y_{sp.sl} - \frac{t_d}{2}\right) \cdot \left(t_d + \frac{h_{so}}{2} - y_{sp.sl}\right)\right] = 47.589\ mm$



$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e_c}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\bar{\lambda}_c - 0.2) + \bar{\lambda}_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is zero -> column like behviour

Interaktion mellan de två olika beteendena(column/plate bucling):

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := \pi_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.65 \times 10^{4}\ kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{\pi_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -336.018\ MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_{sw})}{\gamma_M} = 281.032\ MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_{sw})}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inertia

$A_{d.eff} := n_{so} \cdot s_{so} \cdot t_{so.eff} = 0.032\ m^2$

$A_{tot.eff} := A_{sw.eff} + A_{d.eff} + A_w + A_{bt} + A_b + A_{c.eff} = 0.132\ m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{sw.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \right. \\ &\left. + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{st}}{2}\right)\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{bt} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{wth}) - \frac{h_{wy}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{tt} - \frac{h_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 689.35\ mm$

$I_{y.s.eff} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} \dots + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{st}}{2} - z_{tp.eff.neg}\right)^2\right]\right]\right] = 2.543 \times 10^{-3}\ m^4$

$I_{y.d.eff} := \frac{n_{so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 0.015\ m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s.eff} + I_{y.d.eff} = 0.055\ m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{bf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^{-4}\ m^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.801 \times 10^{3}\ mm^2$

$A_{bt.eff.neg} := \begin{cases} \text{No reduction if the side plate is in class 4.} & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \text{Slenderer class 4 -> no reduction} & \\ \rho_c \cdot A_{c.eff.loc} & \text{otherwise} \end{cases} = 432.173\ cm^2$

$A_{tot.eff.neg} := A_{bf.eff.neg} + A_{sw.eff.neg} + A_w + A_{bt.eff.neg} + A_b = 1.229 \times 10^{3}\ cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.916\ mm$

$y_{neg.comp} := \frac{\begin{aligned} &\pi_{so} \cdot s_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{2} \cdot \left(t_d + \frac{h_{so}}{2}\right) \dots \\ &+ s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + \pi_{so} \cdot s_{so.eff} \cdot t_d \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) \dots \\ &+ t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{st}}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots \\ &+ A_{bt} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{wth}) - \frac{h_{wy}}{2}\right] + A_b \cdot \left(h_{tt} - \frac{h_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 686.683\ mm$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 0.015\ m^4$

$\Delta_{web} := \sin\left(21.12 \cdot \frac{\pi_{so}}{180}\right) \cdot t_{so0} = 7.206\ mm$

Stiffeners over/under:

$I_{y.st.so.fin} := \pi_{so} \cdot \frac{s_{so} \cdot h_{so.eff}^3}{12} + \pi_{so} \cdot s_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 7.453 \times 10^{-3}\ m^4$

$I_{y.st.su.1.fin} = 6 \cdot 1.506 \cdot 10^{-6}\ m^4$ from FEM

$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots + 2 \cdot s_{so} \cdot h_{so} \cdot \left[t_d + h_s + \frac{1}{3} \cdot (h_{wy} - s_{wth}) - \frac{h_{wy}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot s_{so} \cdot h_{so} \cdot \left[t_d + h_s + \frac{2}{3} \cdot (h_{wy} - s_{wth}) - \frac{h_{wy}}{2} - z_{tp.eff.neg}\right]^2 \dots + 2 \cdot s_{so} \cdot h_{so} \cdot \left[t_d + h_s + (h_{wy} - s_{wth}) - \frac{h_{wy}}{2} - z_{tp.eff.neg}\right]^2 = 3.415 \times 10^{-3}\ m^4$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 \cdot t_w \cdot h_w \cdot \left[h_{tt} - \frac{h_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp.eff.neg}\right]^2 = 0.012\ m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot h_b^3}{12} + t_b \cdot h_b \cdot \left(h_{tt} - z_{tp.eff.neg} - \frac{h_b}{2}\right)^2 = 0.011\ m^4$

Side plate:

$I_{y.s.fin} := 2 \cdot \left[\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots + \left[\frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{st}}{2} - z_{tp.eff.neg}\right)^2\right]\right]\right] = 2.525 \times 10^{-3}\ m^4$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 5.152 \times 10^{-10}\ mm^4$

Load

Load capacity:

$q_M := 187 \frac{kN}{m^2}$ The load capacity is calculated with different checks. The load capacity is q_M when checking the moment capacity.

$Q_M := q_M \cdot b_d = 654.5\ \frac{kN}{m}$

$M_{Ed} := \frac{Q_M \cdot l_{bridge}^2}{8} = 2.651 \times 10^4\ kN \cdot m$

$q_V := 155\ \frac{kN}{m^2}$ The load capacity is q_V when checking the shear capacity.

$Q_V := q_V \cdot b_d = 542.5\ \frac{kN}{m}$

$V_{Ed} := \frac{Q_V \cdot l_{bridge}}{2} = 4.883 \times 10^3\ kN$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.075\ m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_fl)}{\gamma_{M1}} = 2.663 \times 10^4\ kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.995$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15st on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot t(t_{wb}) + t_w = 0.293\ m$

$\alpha_w := \operatorname{atan}\left(\frac{h_{web}}{b_w}\right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ss}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{ss}}{2} = 0.503\ m$

$a_{st.z.2} := a_{st.z.1} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\ m$

$a_{st.z.3} := a_{st.z.2} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\ m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 \cdot t_{ss} \cdot h_{ss} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\ m^4$

Distance from z-axis to middle of the web area which is 15st each side of the stiffeners:

$\overline{a_{web.z}} := \frac{b_b}{2} + s_{wall} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{ss}}{2} = 0.525\ m$

$\overline{a_{st.z.1}} := a_{st.z.1} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\ m$

$\overline{a_{st.z.2}} := a_{st.z.2} + s_{ss} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\ m$

Moment of inertia of 15st of the web z-direction

$I_{stw.z} := 2 \cdot h_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021\ m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\ m^4$

Shear buckling coefficient, side plate

$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{otherwise} \end{cases} = 6.247$

$k_{\tau 2} := \max\left(9 \cdot \left(\frac{h_w}{a}\right) \cdot \sqrt{\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right) = 4.782 \times 10^{-3}$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 1.044$ EC1-3-5 eq.5.6

SS-EN 1993-1-5 A.3 (1) och (2) ger:

$k_\tau := \begin{cases} 5.34 + 4 \cdot \left(\frac{h_{web}}{a}\right)^2 + k_{\tau 1} & \text{if } \alpha_w \geq 3 \\ 6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}} + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right) \cdot h_{web}}} & \text{otherwise} \end{cases} = 956.892$ (A5)

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\overline{\gamma_M} := 1.2$ EC1993-1-3-5 ANM.2 5.1

$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min\left(t_s, t_w\right)} \leq 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 \cdot t_w \cdot \varepsilon(t_w) \cdot \sqrt{k_\tau}}, \lambda_{w1}\right) = 1.044$ SS-EN 1993-1-5 5.3 (5.6)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 0.795$ SS-EN 1993-1-5 5.3 Tabell 5.1, stiff end.

Kapacity of the web

$\overline{\gamma_{M0}} := 1.00$ $\overline{\gamma_{M1}} := 1.2$ Safety class 3

$\overline{\gamma_{M0}} := 1.00$ SB3 "bärande konstruktioner"

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \eta} = 2.453 \times 10^3\ kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 4.883 \times 10^3\ kN$

$\frac{V_{Ed}}{2 \cdot V_{bw.Rd}} = 0.995$ (Contribution from two webs)

$\frac{V_{Ed}}{2 \cdot V_{b.Rd}} = \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{tf.eff.neg} := z_{tp.eff.neg} - \frac{t_{fl}}{2} = 681.683\ mm$

$a_{sf.eff.neg} := h_{tt} - \frac{h_b}{2} - z_{tp.eff.neg} = 836.117\ mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{tf.eff.neg} \cdot \left|a_{tf.eff.neg}\right| \cdot f_y(t_{fl})}{\gamma_{M0}} + \frac{A_{sf.eff.neg} \cdot \left|a_{sf.eff.neg}\right| \cdot f_y(t_s)}{\gamma_{M0}} = 1.253 \times 10^4\ kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fulfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{tf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_{fl} \cdot t_{fl}^2 \cdot f_y(t_{fl})}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.541\ m$

$c_{tf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_{fl} \cdot t_{fl}^2 \cdot f_y(t_{fl})}{t_s \cdot \left(h_s + h_{wy}\right)^2 \cdot f_y(t_s)}\right] = 0.53\ m$

$V_{tf.Rd.cred} := \frac{b_{sf} \cdot t_{sf}^2 \cdot f_y(t_{sf})}{c_{tf} \cdot \gamma_{M1} \cdot \eta} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{tf} \cdot \gamma_{M1} \cdot \eta} = 334.405\ kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{tf.Rd} := \begin{cases} V_{tf.Rd.cred} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\ kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{tf.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{w.tot.Rd} := V_{bw.Rd} + V_{tf.Rd} = 2.453 \times 10^3\ kN$

Check:

$\frac{V_{Ed}}{2} \leq 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 \cdot V_{b.Rd}} = 0.995$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 0.026\ m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{se} \cdot s_{so.eff} \cdot t_{d.eff} + n_{se} \cdot t_{se} \cdot t_{d.eff} + A_b = 432.579\ cm^2$

$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{se} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \ldots + n_{se} \cdot t_{se} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_{tt} - \frac{h_b}{2}\right)}{A_{eff.f.tot}} = 565.817\ mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.046\ m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_{fl})}{\gamma_M} = 1.639 \times 10^4\ kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 2.663 \times 10^4\ kN \cdot m$

Moment capacity check when $q_M = 187\ \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.995$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

Shear capacity check when $q_V = 155\ \frac{kN}{m^2}$

$\eta_3 := \frac{V_{Ed}}{2 \cdot V_{bw.Rd}} = 0.995$

The Reduced stress method

-Changed side plate height to 1030 mm

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ 335\text{MPa} & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\upsilon := 0.3$ $G := \frac{E}{2\cdot(1+\upsilon)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate:

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 10\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d t_d = 0.035\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 1030\text{mm}$ $A_s := h_s t_s = 0.012\,\text{m}^2$

Webs:

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w t_w = 0.016\,\text{m}^2$

$b_{w.x} := 1250\text{mm}$ $b_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{st} := 16\text{mm}$ $h_{st} := 140\text{mm}$ $A_{st} := h_{st} t_{st} = 2.24\times10^{-3}\,\text{m}^2$

$x_{st} := 350\text{mm}$ $n_{st} := 9$

Web stiffeners:

$t_{wt} := 12\text{mm}$ $h_{wt} := 120\text{mm}$ $A_{wt} := h_{wt} t_{wt} = 1.44\times10^{-3}\,\text{m}^2$

$x_{wt} := 440\text{mm}$ $n_{wt} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{h_s}{2} + h_{w.y} + h_s + t_d = 1.531\,\text{m}$ $x_{ts} := 2\text{m}$

$I_{tot} := 0.053092\text{m}^4$ From grasshopper

$Y_{tp} := 0.861542\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 12.4\,\frac{\text{kN}}{\text{m}^2}$

$Q := q\,b_d = 434\,\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.758\times10^4\,\text{kN·m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 3.906\times10^3\,\text{kN}$

Top plate:

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 219.914\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\,b_d}{n_{st}+2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 2.114\times10^{-3}\,\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\,S_{tp}}{I_{tot}\cdot\min(t_s, t_{wt})} = 12.958\cdot\text{MPa}$

Side plates:

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts} - t_d - Y_{tp}\right) = 218.258\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2\,t_s} = 18.083\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)t_s\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\,b_d}{2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 0.02\,\text{m}^3$

$\sigma_{x.Ed.sps} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts} - t_d - Y_{tp}\right) = 218.258\cdot\text{MPa}$

$S_{tps} := \left(n_{st}\frac{t_{st}\,h_{st}}{2} + 2\frac{t_s\,h_{wt}}{2}\right)\left(h_{ts} - t_d - \frac{h_{st}}{4} - Y_{tp}\right) = 2.781\times10^{-3}\,\text{m}^3$
$+ \frac{t_d\,b_d}{n_{st}+2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\,S_{tps}}{I_{tot}\,t_{wt}} = 12.787\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 2.53\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Solidik

4.2449

$\sigma_V := \sigma_{V.UNIT}\,q = 313.72\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.132$

$\alpha_{cr.UNIT} := 211.64\,\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Solidik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 1.707$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.814$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.896$

Plate-like behaviour: [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot\left(1-\upsilon^2\right)\cdot b_d^2} = 1.549\cdot\text{MPa}$

$I_{y.tp} := \frac{b_d\cdot t_d^3}{12\cdot\left(1-\upsilon^2\right)} = 3.205\times10^{-7}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\frac{t_d}{2} + n_{st}\cdot A_{st}\left(t_d + \frac{h_{st}}{2}\right)}{A_d + n_{st}\cdot A_{st}} = 32.411\cdot\text{mm}$

$I_{sl.tp} := \frac{b_d\,t_d^3}{12} + A_d\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 + \ldots = 1.052\times10^{-6}\,\text{mm}^4$
$+ n_{st}\left[\frac{t_{st}\,h_{st}^3}{12} + A_{st}\left(t_d + \frac{h_{st}}{2} - y_{tp.sl.tp}\right)^2\right]$

$n_{st} = 9$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{y.tp}} = 328.143$

$\delta_{tp} := \frac{n_{st}\cdot A_{st}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\sqrt{\frac{\gamma_{tp}}{1+\delta_{tp}}}, 0.5\right) = 0.571$ $\sqrt{\gamma_{tp}} = 4.256$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\left(\psi_{tp}+1\right)\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} \le \sqrt{\gamma_{tp}} \\ \frac{4\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{tp}+1\right)\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} > \sqrt{\gamma_{tp}} \end{cases} = 639.127$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\,\sigma_{E.tp} = 990.256\cdot\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\,x_{tst} = 0.175\,\text{m}$

$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\,x_{tst} = 0.175\,\text{m}$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,\text{m}$

$y_{tp.sl.1.tp} := \frac{t_d\,b_{sl.1.tp}\frac{t_d}{2} + A_{st}\left(t_d + \frac{h_{st}}{2}\right)}{t_d\,b_{sl.1.tp} + A_{st}} = 0.034\,\text{m}$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^3}{12} + b_{sl.1.tp}\,t_d\left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{st}\,h_{st}^3}{12} + A_{st}\left(t_d + \frac{h_{st}}{2} - y_{tp.sl.1.tp}\right)^2$

Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot x_{tst}^2} = 1.026\times10^3\cdot\text{MPa}$ critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,\text{m}$

$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d - t_d + \frac{h_{st}}{2} - y_{tp.sl.1.tp}\right) = 0.046\,\text{m}$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e.tp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 1.01$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\left(\phi_{tp} + \sqrt{\phi_{tp}^2 - \lambda_p^2}\right)}, 1\right) = 0.622$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp} - \chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right) + \chi_{tp}\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.622$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.019$

$\frac{\rho_{c.tp}\,\alpha_{ult}}{\gamma_{M1}} = 0.703$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}\,z + \sigma_{max}\right]\cdot1$

$\psi_{sp.x} := \frac{\sigma(h_{ts} - t_d - h_s)}{\sigma(h_{ts} - t_d)} = -0.562$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{sp.x})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1.026$

Plate-like behaviour

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts} - t_d - h_s - Y_{tp}\right) = -122.74\cdot\text{MPa}$

$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\upsilon^2\right)\cdot h_s^2} = 25.762\cdot\text{MPa}$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.sup}} = -0.562$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} \equiv 1 \\ \frac{8.2}{\left(1.05 + \psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} \equiv 0 \\ 7.81 - 6.29\cdot\psi_{sp.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} \equiv -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} > -3 \\ 85.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 14.44$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\,k_{\sigma.sp.x} = 372.013\cdot\text{MPa}$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot i_s^2}{12\cdot\left(1-\upsilon^2\right)\cdot x_{tst}^2} = 6.833\cdot\text{MPa}$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp}\left(\lambda_p - 0.2\right) + \lambda_p^2\right] = 0.896$

$\chi_{sp} := \min\left(\frac{1}{\left(\phi_{sp} + \sqrt{\phi_{sp}^2 - \lambda_p^2}\right)}, 1\right) = 0.787$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) + \chi_{sp}\right]\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right) + \chi_{sp}\right]\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} := 1$

$\sigma_{p.sp} := 0.34$ $\lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \sigma_{p.sp}\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.927$

$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 0.88$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\upsilon^2\right)\cdot x_{tst}^2} = 6.833\cdot\text{MPa}$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\,\sigma_{E.sp.z} = 27.331\cdot\text{MPa}$

Column-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot i_s^2}{12\cdot\left(1-\upsilon^2\right)\cdot h_s^2} = 25.762\cdot\text{MPa}$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0.061$

$$\rho_{c,sp,z} := \begin{cases} \left[\left(\rho_{pz,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2 - \xi_{sp,z}\right) + \chi_{sp}\right] & \text{if } \frac{\left[\left(\rho_{pz,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2 - \xi_{sp,z}\right) + \chi_{sp}\right]\sigma_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{pz,sp} - \chi_{sp}\right)\xi_{sp,z}\left(2 - \xi_{sp,z}\right) + \chi_{sp}\right]\sigma_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.798$$

Top plate stiffeners - check

x-direction

$$\rho_{px,tps} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 0.945$$

$$\psi_{tps,x} := \frac{\sigma\left(h_w - t_d - h_{na}\right)}{\sigma\left(h_w - t_d\right)} = 0.788$$

$$\sigma_{E,tps,x} := \frac{\pi^2 \cdot E \cdot t_{sp}^{\ 2}}{12\left(1 - \nu^2\right)b_{sp}^{\ 2}} = 2.479 \times 10^3\ \text{MPa}$$

$$k_{\sigma,tps,x} := \begin{cases} 1 & \text{if } \psi_{tps,x} \equiv 1 \\ \frac{8.2}{\left(1.05 + \psi_{tps,x}\right)} & \text{if } 1 > \psi_{sp,x} > 0 \\ 7.81 & \text{if } \psi_{tps,x} \equiv 0 \\ 7.81 - 6.29 \cdot \psi_{tps,x} + 9.78 \cdot \psi_{sp,x}^{\ 2} & \text{if } 0 > \psi_{sp,x} > -1 \\ 23.9 & \text{if } \psi_{tps,x} \equiv -1 \\ 5.98\left(1 - \psi_{tps,x}\right)^2 & \text{if } -1 > \psi_{tps,x} > -3 \\ 85.68 & \text{if } \psi_{tps,x} < -3 \end{cases} = 5.949$$

$$\sigma_{cr,p,tps,x} := k_{\sigma,tps,x}\sigma_{E,tps,x} = 1.475 \times 10^4\ \text{MPa}$$

Column-like behaviour

$$\sigma_{cr,c,tps,x} := \frac{\pi^2 \cdot E \cdot t_{sp}^{\ 2}}{12\left(1 - \nu^2\right)\cdot s_{ts}^{\ 2}} = 12.147\ \text{MPa}$$

$$\chi_{tps} := \chi_{sp}$$

Interpolation

$$\xi_{tps,x} := \min\left(\max\left(\frac{\sigma_{cr,p,tps,x}}{\sigma_{cr,c,tps,x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c,tps,x} := \begin{cases} \left[\left(\rho_{px,tps} - \chi_{tps}\right)\xi_{tps,x}\left(2 - \xi_{tps,x}\right) + \chi_{tps}\right] & \text{if } \frac{\left[\left(\rho_{px,tps} - \chi_{tps}\right)\xi_{tps,x}\left(2 - \xi_{tps,x}\right) + \chi_{tps}\right]\sigma_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px,tps} - \chi_{tps}\right)\xi_{tps,x}\left(2 - \xi_{tps,x}\right) + \chi_{tps}\right]\sigma_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_{sp} \cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.997$$

Side plate check

$$\left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sp,z} \cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x,Ed,sp}}{\frac{\rho_{c,sp,x} \cdot f_y(t_s)}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z,Ed,sp}}{\frac{\rho_{c,sp,z} \cdot f_y(t_s)}{\gamma_{M1}}}\right) + 3\left(\frac{\tau_{Ed,sp}}{\frac{\chi_{sp} \cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.43$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x,Ed,tps}}{\frac{\rho_{c,tps,x} \cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed,tps}}{\frac{\chi_{sp} \cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.382$$

C.6 Changed vertical stiffener distance

CCCXI RSM: 1.73m vertical stiffener distance.
CCCXV RSM: 1.72 mm vertical stiffener distance.
CCCXIX RSM: 1.636m vertical stiffener distance.
CCCXXII RSM: 1.385m vertical stiffener distance.
CCCXXVI RSM: 2m vertical stiffener distance.
CCCXXX RSM: 2.25m vertical stiffener distance.
CCCXXXIV RSM: 2.57m vertical stiffener distance.
CCCXXXVIII RSM: 3m vertical stiffener distance.
CCCXLII RSM: 3.35m vertical stiffener distance.

CCCXLVIII EWM: 1.6m vertical stiffener distance.
CCCLIV EWM: 1.8m vertical stiffener distance.
CCCLX EWM: 2m vertical stiffener distance.
CCCLXVI EWM: 2.2m vertical stiffener distance.
CCCLXXII EWM: 2.4m vertical stiffener distance.
CCCLXXVIII EWM: 2.6m vertical stiffener distance.
CCCLXXXIV EWM: 2.8m vertical stiffener distance.
CCCXC EWM: 3m vertical stiffener distance.
CCCXCVI EWM: 3.2m vertical stiffener distance.
CDII EWM: 3.4m vertical stiffener distance.

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b \cdot t_b = 0.016\text{m}^2$

Top plate

$t_d := 10\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d \cdot t_d = 0.035\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s \cdot t_s = 4.67\times10^{-3}\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w \cdot t_w = 0.016\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so} \cdot t_{so} = 2.24\times10^{-3}\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su} \cdot t_{su} = 1.44\times10^{-3}\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $s_{ts} := 1.5\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\text{m}$

$I_{tot} := 0.0148\text{m}^4$ From grasshopper

$Y_{tp} := 0.526956\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 78\frac{\text{kN}}{\text{m}^2}$

$Q := q \cdot b_d = 273.\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.106\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.457\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 267.481\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 15.761\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 11.375\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5 V_{Ed}\cdot S_{sp}}{I_{tot}\cdot t_s} = 68.25\cdot\text{MPa}$

Top plate stifeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 15.348\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.33\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_V := \sigma_{V.UNIT}\cdot q = 337.74\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.051$

$\alpha_{cr.UNIT} := 230.11\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.95$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.597$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\left(1-\nu^2\right)\cdot b_d^2} = 1.549\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\left(1-\nu^2\right)} = 3.205\times10^{-7}\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\cdot A_{so}} = 32.411\cdot\text{mm}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\cdot\text{mm}^4$
$+ n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.5$

$\sqrt[4]{\gamma_{tp}} = 4.256$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\cdot\left[\left(1+\alpha_{tp}^{\,2}\right)^{2}+\gamma_{tp}-1\right]}{\alpha_{tp}^{\,2}\cdot\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 834.277$$

$$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 1.293\times 10^{3}\cdot MPa$$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

$$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$$

$$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2}+A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)^{-3}\,?}{t_d\cdot b_{sl.1.tp}+A_{so}} = 0.034\,m$$

combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^{\,3}}{12}+b_{sl.1.tp}\cdot t_d\cdot\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^{2}+\frac{t_{so}\cdot h_{so}^{\,3}}{12}+A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^{2}$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^{2}\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}^{\,2}} = 1.825\times 10^{3}\cdot MPa$$

critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,m$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp}-t_d\cdot t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right) = 0.046\,m$$

$$\alpha_{e.tp} := 0.49+\frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5\left[1+\alpha_{e.tp}\cdot\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 0.794$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}^{\,2}-\lambda_p^{\,2}}},1\right) = 0.759$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}}-1,0\right),1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right] & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.759$$

Shear

$$\eta := 1.2$$

7.1 (2)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \dfrac{0.83}{\eta} \\ \dfrac{0.83}{\lambda_p} & \text{if } \dfrac{0.83}{\eta} \le \lambda_p < 1.08 \\ \dfrac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$$

$$\frac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.798$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}\cdot z+\sigma_{max}\right]-1$$

$$\psi_{sp.x} := \frac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = -0.102$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-Y_{tp}\right) = 263.745\cdot MPa$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-h_s-Y_{tp}\right) = -27.011\cdot MPa$$

$$\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$$

$$\psi_{sp3.x} := \max\left(3+\psi_{sp.x},0\right) = 2.898$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p-0.055\cdot\left(\psi_{sp3.x}\right)}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^{2}\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^{2}\right)\cdot h_s^{\,2}} = 180.432\cdot MPa$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \dfrac{8.2}{\left(1.05+\psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81-6.29\cdot\psi_{sp.x}+9.78\cdot\psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^{2} & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.557$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.544\times 10^{3}\cdot MPa$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^{2}\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^{2}\right)\cdot s_{ts}^{\,2}} = 12.147\cdot MPa$$

$$\alpha_{sp} := 0.21$$

$$\phi_{sp} := 0.5\left[1+\alpha_{sp}\cdot\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 0.72$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp}+\sqrt{\phi_{sp}^{\,2}-\lambda_p^{\,2}}},1\right) = 0.891$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}}-1,0\right),1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)+\chi_{sp}\right] & \text{if } \dfrac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)+\chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)+\chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.781$

$$\rho_{pz.sp} := \begin{cases} 1 \text{ if } \lambda_p \le 0.673 \\ \dfrac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} \text{ if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_{ts}^2} = 12.147\cdot MPa$$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 48.589\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\dfrac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] \text{ if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 \text{ if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.891$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \le 0.748 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^2} \text{ if } \lambda_p > 0.748 \end{cases} = 1$$

$\psi_{tps.x} := \dfrac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$

$$\sigma_{E.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$$

$$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 \text{ if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 \text{ if } \psi_{tps.x} < -3 \end{cases} = 4.117$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.021\times 10^4\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot h_{ts}^2} = 21.595\cdot MPa$$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\dfrac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right] \text{ if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 \text{ if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.989$$

$$3\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 4.106\times 10^{-3}$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.985$$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\xi_{tp}\left(2 - \xi_{tp}\right) + \chi_{tp} = 0.759$

$c_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.33$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp.V}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^{c_3}} = 0.993$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y(t_s)}{\gamma_{M1}}}\right) \ldots = 0.604$$
$$+ 3\cdot\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2$$

With the V-factor

$c_{sp} := \min\left(1.25 - 0.25\cdot\psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right)\cdot\xi_{sp.x}\left(2 - \xi_{sp.x}\right) + \chi_{sp} = 1$

$c_3 := 1.25 + 0.75\cdot\chi_w^2 = 2.33$

$V_{sp} := \rho_{c.sp.x.V}\cdot\rho_{c.sp.z} = 0.891$

$f_{sp} := \lambda_p^{-0.5\cdot\ln\left(\xi_{sp.z} + 1\right)^{0.9}} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right)\cdot f_{sp} = 0.891$

$$\sqrt{\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 \ldots} = 0.762$$
$$\ldots + -V_{sp}\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right) + \left(\frac{\sqrt{3}\cdot\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{c_3}$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.556$$

$$\varepsilon_{tps} := \min\left(1.25 - 0.25\cdot\psi_{tps.x}, 1.25\right) = 1.099$$

$$\rho_{tps.x.V} := \min\left[\varepsilon_{tps}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$$

$$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V} - \chi_{tps}\right)\cdot\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) + \chi_{tps} = 1$$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}} = 0.744$$

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 10\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.035\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge $s_{ts} := 1.8\text{m}$

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\,\text{m}$

$I_{tot} := 0.0148\text{m}^4$ From grasshopper

$Y_{tp} := 0.526956\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 78\,\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 273\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.106\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.457\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 267.481\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 15.761\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 11.375\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\,t_s} = 68.25\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 15.348\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.37\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_V := \sigma_{V.UNIT}\cdot q = 340.86\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.041$

$\alpha_{cr.UNIT} := 231.41\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.967$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.592$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-7}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\,A_{so}} = 32.411\cdot\text{mm}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\cdot\text{mm}^4$
$+ n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.514$ $\sqrt[4]{\gamma_{tp}} = 4.256$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\cdot\left[\left(1+\alpha_{tp}{}^2\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}{}^2\cdot\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 788.66$$

$$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 1.222\times10^{3}\cdot\text{MPa}$$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\ \text{m}$$

$$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\ \text{m}$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\ \text{m}$$

$$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\cdot b_{sl.1.tp}+A_{so}} = 0.034\ \text{m}$$

combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d{}^3}{12} + b_{sl.1.tp}\cdot t_d\cdot\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \frac{t_{so}\cdot h_{so}{}^3}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}{}^2} = 1.267\times10^{3}\cdot\text{MPa}$$

critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\ \text{m}$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp}-t_d,\ t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right) = 0.046\ \text{m}$$

$$\alpha_{e.tp} := 0.49+\frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5\cdot\left[1+\alpha_{e.tp}\cdot\left(\lambda_p-0.2\right)+\lambda_p{}^2\right] = 0.79$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}{}^2-\lambda_p{}^2}},1\right) = 0.762$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}}-1,0\right),1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right] & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.762$$

Shear

$$\eta := 1.2 \qquad 7.1\ (2)$$

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$$

$$\frac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.794$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}\cdot z+\sigma_{max}\right]\cdot 1$$

$$\psi_{sp.x} := \frac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = -0.102$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-Y_{tp}\right) = 263.745\cdot\text{MPa}$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-h_s-Y_{tp}\right) = -27.011\cdot\text{MPa}$$

$$\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$$

$$\psi_{sp3.x} := \max\left(3+\psi_{sp.x},0\right) = 2.898$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p-0.055\cdot\left(\psi_{sp3.x}\right)}{\lambda_p{}^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s{}^2}{12\cdot\left(1-\nu^2\right)\cdot h_s{}^2} = 180.432\cdot\text{MPa}$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \dfrac{8.2}{\left(1.05+\psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81-6.29\cdot\psi_{sp.x}+9.78\cdot\psi_{sp.x}{}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.557$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.544\times10^{3}\cdot\text{MPa}$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot t_s{}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}{}^2} = 8.436\cdot\text{MPa}$$

$$\alpha_{sp} := 0.21$$

$$\phi_{sp} := 0.5\cdot\left[1+\alpha_{sp}\cdot\left(\lambda_p-0.2\right)+\lambda_p{}^2\right] = 0.717$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp}+\sqrt{\phi_{sp}{}^2-\lambda_p{}^2}},1\right) = 0.893$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}}-1,0\right),1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)\cdots\right] & \text{if } \dfrac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)\cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)+\chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\cdot\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.778$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \dfrac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 8.436\cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 33.742\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \dfrac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\dfrac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] + \chi_{sp} & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.893$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$\psi_{tps.x} := \dfrac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$

$\sigma_{E.tps.x} := \dfrac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 & \text{if } \psi_{tps.x} = 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{tps.x} < -3 \end{cases} = 4.117$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.021\times 10^4\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \dfrac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 14.997\cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\dfrac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] + \chi_{tps} & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases}$$

Top plate check

$\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\dfrac{\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.982$

$3\left(\dfrac{\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 4.106\times 10^{-3}$

$\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.977$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\xi_{tp}\cdot\left(2 - \xi_{tp}\right) + \chi_{tp} = 0.762$

$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.33$

$\sqrt{\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp.V}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + \left(\dfrac{\sqrt{3}\cdot\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^{e_3}} = 0.989$

Side plate check

$\left(\dfrac{\sigma_{x.Ed.sp}}{\dfrac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\dfrac{\sigma_{z.Ed.sp}}{\dfrac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\dfrac{\sigma_{x.Ed.sp}}{\dfrac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)\cdot\left(\dfrac{\sigma_{z.Ed.sp}}{\dfrac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.604$

$+ 3\cdot\left(\dfrac{\tau_{Ed.sp}}{\dfrac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$

With the V-factor

$c_{sp} := \min\left(1.25 - 0.25\cdot\psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right)\cdot\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) + \chi_{sp} = 1$

$e_{3.V} := 1.25 + 0.75\cdot\chi_w^2 = 2.33$

$V_{sp} := \rho_{c.sp.x.V}\cdot\rho_{c.sp.z} = 0.893$

$f_{sp} := \lambda_p^{-0.5}\cdot\ln\left(\xi_{sp.z} + 1\right)^{0.9} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right)\cdot f_{sp} = 0.893$

$\sqrt{\left(\dfrac{\sigma_{x.Ed.sp}}{\dfrac{\rho_{c.sp.x.V}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\dfrac{\sigma_{z.Ed.sp}}{\dfrac{\rho_{c.sp.z.V}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - V_{sp}\cdot\left(\dfrac{\sigma_{x.Ed.sp}}{\dfrac{\rho_{c.sp.x.V}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)\cdot\left(\dfrac{\sigma_{z.Ed.sp}}{\dfrac{\rho_{c.sp.z.V}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)} \cdots = 0.762$

$+ \left(\dfrac{\sqrt{3}\cdot\tau_{Ed.sp}}{\dfrac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^{e_3}$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.556$$

$$c_{tps} := \min\left(1.25 - 0.25\cdot\psi_{tps.x}, 1.25\right) = 1.099$$

$$\rho_{tps.x.V} := \min\left[c_{tps}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 1$$

$$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V} - \chi_{tps}\right)\cdot\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) + \chi_{tps} = 1$$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}} = 0.744$$

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ (335\,MPa) & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} := 1$

$E := 210\,GPa$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$t_b := 16\,mm$ $b_b := 1000\,mm$ $A_b := b_b \cdot t_b = 0.016\,m^2$

Top plate

$t_d := 10\,mm$ $b_d := 3500\,mm$ $A_d := b_d \cdot t_d = 0.035\,m^2$

Side plates

$t_s := 12\,mm$ $h_s := 389.2\,mm$ $A_s := h_s \cdot t_s = 4.67\times10^{-3}\,m^2$

Webs

$t_w := 12\,mm$ $h_w := 1340\,mm$ $A_w := h_w \cdot t_w = 0.016\,m^2$

$h_{w.x} := 1250\,mm$ $h_{w.y} := 482.8\,mm$

Top stiffeners

$t_{so} := 16\,mm$ $h_{so} := 140\,mm$ $A_{so} := h_{so} \cdot t_{so} = 2.24\times10^{-3}\,m^2$

$s_{so} := 350\,mm$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\,mm$ $h_{su} := 120\,mm$ $A_{su} := h_{su} \cdot t_{su} = 1.44\times10^{-3}\,m^2$

$s_{su} := 440\,mm$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\,m$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\,m$

$s_{ts} := \frac{18}{11}\,m = 1.636\,m$

$I_{tot} := 0.0148\,m^4$ From grasshopper

$Y_{tp} := 0.526956\,m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 78\,\frac{kN}{m^2}$

$Q := q \cdot b_d = 273.\,\frac{kN}{m}$

$M_{Ed} := \frac{Q \cdot L_{bridge}^2}{8} = 1.106\times10^4\,kN\cdot m$

$V_{Ed} := \frac{Q \cdot L_{bridge}}{2} = 2.457\times10^3\,kN$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 267.481\,MPa$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d \cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\,m^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 15.761\,MPa$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot(h_{ts} - t_d - Y_{tp}) = 263.745\,MPa$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 11.375\,MPa$

$S_{sp} := (h_{ts} - t_d - Y_{tp})\cdot t_s\cdot\frac{(h_{ts} - t_d - Y_{tp})}{2} + \frac{t_d \cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\,m^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5 V_{Ed}\cdot S_{sp}}{I_{tot}\cdot t_s} = 68.25\,MPa$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\,MPa$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\,m^3$
$+ \frac{t_d\, b_d}{n_{so}+2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 15.348\,MPa$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.57\,\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_V := \sigma_{V.UNIT}\cdot q = 356.46\,MPa$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 0.996$

$\alpha_{cr.UNIT} := 214.1\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.745$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.602$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\,MPa$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-7}\,m^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\cdot A_{so}} = 32.411\,mm$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\,mm^4$
$+ n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.5$ $\sqrt[4]{\gamma_{tp}} = 4.256$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\cdot\left[\left(1+\alpha_{tp}{}^2\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}{}^2\cdot\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp}\le\sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp}>\sqrt[4]{\gamma_{tp}} \end{cases} = 834.277$$

$$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 1.293\times10^{3}\cdot MPa$$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

$$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$$

$$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2}+A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\cdot b_{sl.1.tp}+A_{so}} = 0.034\,m$$

combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d{}^3}{12}+b_{sl.1.tp}\cdot t_d\cdot\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2+\frac{t_{so}\cdot h_{so}{}^3}{12}+A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}{}^2} = 1.533\times10^{3}\cdot MPa$$

critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,m$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp}-t_d\cdot t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right) = 0.046\,m$$

$$\alpha_{e.tp} := 0.49+\frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5\left[1+\alpha_{e.tp}\cdot\left(\lambda_p-0.2\right)+\lambda_p{}^2\right] = 0.799$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}{}^2-\lambda_p{}^2}},1\right) = 0.756$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}}-1,0\right),1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}}<1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}}\ge1 \end{cases} = 0.756$$

Shear

$\eta := 1.2$ 7.1 (2)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p<\frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta}\le\lambda_p<1.08 \\ \frac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p\ge1.08 \end{cases} = 1.2$$

$$\frac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.753$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}\cdot z+\sigma_{max}\right]-1$$

$$\psi_{sp.x} := \frac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = -0.102$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts}-t_d-Y_{tp}\right) = 263.745\cdot MPa$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts}-t_d-h_s-Y_{tp}\right) = -27.011\cdot MPa$$

$$\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$$

$$\psi_{sp3.x} := \max\left(3+\psi_{sp.x},0\right) = 2.898$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p\le0.673 \\ \frac{\lambda_p-0.055\cdot\left(\psi_{sp3.x}\right)}{\lambda_p{}^2} & \text{if } \lambda_p>0.673 \end{cases} = 1$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s{}^2}{12\cdot\left(1-\nu^2\right)\cdot h_s{}^2} = 180.432\cdot MPa$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x}=1 \\ \frac{8.2}{\left(1.05+\psi_{sp.x}\right)} & \text{if } 1>\psi_{sp.x}>0 \\ 7.81 & \text{if } \psi_{sp.x}=0 \\ 7.81-6.29\cdot\psi_{sp.x}+9.78\cdot\psi_{sp.x}{}^2 & \text{if } 0>\psi_{sp.x}>-1 \\ 23.9 & \text{if } \psi_{sp.x}=-1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1>\psi_{sp.x}\ge-3 \\ 95.68 & \text{if } \psi_{sp.x}<-3 \end{cases} = 8.557$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.544\times10^{3}\cdot MPa$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot t_s{}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}{}^2} = 10.207\cdot MPa$$

$\alpha_{sp} := 0.21$

$$\phi_{sp} := 0.5\left[1+\alpha_{sp}\cdot\left(\lambda_p-0.2\right)+\lambda_p{}^2\right] = 0.724$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp}+\sqrt{\phi_{sp}{}^2-\lambda_p{}^2}},1\right) = 0.889$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}}-1,0\right),1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)\ldots\right]+\chi_{sp} & \text{if } \frac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)\ldots\right]+\chi_{sp}\cdot\alpha_{ult}}{\gamma_{M1}}<1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)+\chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}}\ge1 \end{cases} = 1$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.785$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \dfrac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 10.207\cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 40.828\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \dfrac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\dfrac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) \cdots\right] & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) \cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{sp} & \\ 1 & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.889$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$\psi_{tps.x} := \dfrac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$

$\sigma_{E.tps.x} := \dfrac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 & \text{if } \psi_{tps.x} = 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{tps.x} < -3 \end{cases} = 4.117$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.021\times 10^4\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \dfrac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 18.146\cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\dfrac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \cdots\right] & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{tps} & \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Top plate check

$\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\dfrac{\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.998$

$3\left(\dfrac{\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 4.106\times 10^{-3}$

$\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.994$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\xi_{tp}\cdot\left(2 - \xi_{tp}\right) + \chi_{tp} = 0.756$

$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.33$

$\sqrt{\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp.V}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + \left(\dfrac{\sqrt{3}\cdot\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^{e_3}} = 0.998$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\dfrac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \cdots\right] & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{tps} & \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Top plate check

$\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\dfrac{\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.998$

$3\left(\dfrac{\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 4.106\times 10^{-3}$

$\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.994$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\xi_{tp}\cdot\left(2 - \xi_{tp}\right) + \chi_{tp} = 0.756$

$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.33$

$\sqrt{\left(\dfrac{\sigma_{x.Ed.tp}}{\dfrac{\rho_{c.tp.V}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + \left(\dfrac{\sqrt{3}\cdot\tau_{Ed.tp}}{\dfrac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^{e_3}} = 0.998$

Side plate check

$\left(\dfrac{\sigma_{x.Ed.sp}}{\dfrac{\rho_{c.sp.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\dfrac{\sigma_{z.Ed.sp}}{\dfrac{\rho_{c.sp.z}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 - \left(\dfrac{\sigma_{x.Ed.sp}}{\dfrac{\rho_{c.sp.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\left(\dfrac{\sigma_{z.Ed.sp}}{\dfrac{\rho_{c.sp.z}\cdot f_y(t_s)}{\gamma_{M1}}}\right) \cdots = 0.603$

$+ 3\cdot\left(\dfrac{\tau_{Ed.sp}}{\dfrac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2$

With the V-factor

$c_{sp} := \min\left(1.25 - 0.25\cdot\psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right)\cdot\xi_{sp.x}\left(2 - \xi_{sp.x}\right) + \chi_{sp} = 1$

$e_3 := 1.25 + 0.75\cdot\chi_w^2 = 2.33$

$V_{sp} := \rho_{c.sp.x.V}\cdot\rho_{c.sp.z} = 0.889$

$\xi_{sp} := \lambda_p - 0.5\cdot\ln\left(\xi_{sp.z} + 1\right)^{0.9} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right)\cdot\xi_{sp} = 0.889$

$\sqrt{\left(\dfrac{\sigma_{x.Ed.sp}}{\dfrac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\dfrac{\sigma_{z.Ed.sp}}{\dfrac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 - V_{sp}\left(\dfrac{\sigma_{x.Ed.sp}}{\dfrac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\left(\dfrac{\sigma_{z.Ed.sp}}{\dfrac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right) \cdots} = 0.762$

$+ \left(\dfrac{\sqrt{3}\cdot\tau_{Ed.sp}}{\dfrac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.556$$

$c_{tps} := \min\left(1.25 - 0.25\cdot\psi_{tps.x}, 1.25\right) = 1.099$

$\rho_{tps.x.V} := \min\left[c_{tps}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^{\,2}}\right), 1\right] = 1$

$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V} - \chi_{tps}\right)\cdot\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) + \chi_{tps} = 1$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2}^{e_3} = 0.744$$

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\text{m}^2$

Top plate

$t_d := 10\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.035\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge $L_{bridge} := 18\text{m}$

$s_{ts} := \frac{18}{13}\text{m} = 1.385\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\text{m}$

$I_{tot} := 0.0148\text{m}^4$ From grasshopper

$Y_{tp} := 0.526956\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 78\cdot\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 273.\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.106\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.457\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 267.481\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 15.761\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 11.375\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5 V_{Ed}\cdot S_{sp}}{I_{tot}\cdot t_s} = 68.25\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 15.348\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.26\cdot\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT}\cdot q = 332.28\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.068$

$\alpha_{cr.UNIT} := 232.72\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.984$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.598$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-7}\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}A_{so}} = 32.411\cdot\text{mm}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\cdot\text{mm}^4$
$+ n_{so}\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.5$

$k_{\sigma.p.tp} := \begin{cases} \frac{2\left[\left(1+\alpha_{tp}^2\right)^2 + \gamma_{tp} - 1\right]}{\alpha_{tp}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \frac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 834.277$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 1.293\times10^3\cdot\text{MPa}$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \frac{3 - \psi_{tp}}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$

$b_{2.sup.tp} := \frac{2}{5 - \psi_{tp}} \cdot s_{so} = 0.175\,m$

Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$

$y_{tp.sl.1.tp} := \frac{t_d \cdot b_{sl.1.tp} \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right)^{-3}}{t_d \cdot b_{sl.1.tp} + A_{so}} = 0.034\,m$

Combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \frac{b_{sl.1.tp} \cdot t_d^{3}}{12} + b_{sl.1.tp} \cdot t_d \cdot \left(y_{tp.sl.1.tp} - \frac{t_d}{2}\right)^2 + \frac{t_{so} \cdot h_{so}^{3}}{12} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right)^2$

Combined moment of inertia

$\sigma_{cr.sl.tp} := \frac{\pi^2 \cdot E \cdot I_{sl.1.tp}}{A_{sl.1.tp} \cdot s_{ts}^{2}} = 2.142 \times 10^{3}\,MPa$

Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,m$

$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d \cdot t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.046\,m$

$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{}} = 0.582$ stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1 + \alpha_{e.tp} \cdot (\lambda_p - 0.2) + \lambda_p^{2}\right] = 0.795$ [6.3.1.2]

$\chi_{tp} := \min\left(\frac{1}{\phi_{tp} + \sqrt{\phi_{tp}^{2} - \lambda_p^{2}}}, 1\right) = 0.758$

Interpolation

$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \left[(\rho_{px.tp} - \chi_{tp}) \cdot \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp}) \cdot \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[(\rho_{px.tp} - \chi_{tp}) \cdot \xi_{tp} \cdot (2 - \xi_{tp}) + \chi_{tp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.758$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \leq \lambda_p < 1.08 \\ \frac{1.37}{(0.7 + \lambda_p)} & \text{if } \lambda_p \geq 1.08 \end{cases} = 1.2$

$\frac{\rho_{c.tp} \cdot \alpha_{ult}}{\gamma_{M1}} = 0.81$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})} \cdot z + \sigma_{max}\right] \cdot 1$

$\psi_{sp.x} := \frac{\sigma(h_{ts} - t_d - h_s)}{\sigma(h_{ts} - t_d)} = -0.102$

$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - Y_{tp}) = 263.745 \cdot MPa$

$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - h_s - Y_{tp}) = -27.011 \cdot MPa$

$\psi_{sp3.x} := \max(3 + \psi_{sp.x}, 0) = 2.898$ $\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{\lambda_p - 0.055 \cdot (\psi_{sp3.x})}{\lambda_p^{2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$\sigma_{E.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{2}}{12 \cdot (1 - \nu^2) \cdot h_s^{2}} = 180.432 \cdot MPa$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \frac{8.2}{(1.05 + \psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{sp.x} + 9.78 \cdot \psi_{sp.x}^{2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98 \cdot (1 - \psi_{sp.x})^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.557$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x} \cdot k_{\sigma.sp.x} = 1.544 \times 10^{3}\,MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \frac{\pi^2 \cdot E \cdot t_s^{2}}{12 \cdot (1 - \nu^2) \cdot s_{ts}^{2}} = 14.256 \cdot MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1 + \alpha_{sp} \cdot (\lambda_p - 0.2) + \lambda_p^{2}\right] = 0.721$

$\chi_{sp} := \min\left(\frac{1}{\phi_{sp} + \sqrt{\phi_{sp}^{2} - \lambda_p^{2}}}, 1\right) = 0.891$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \left[(\rho_{px.sp} - \chi_{sp}) \cdot \xi_{sp.x} \cdot (2 - \xi_{sp.x}) \cdots \right. & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp}) \cdot \xi_{sp.x} \cdot (2 - \xi_{sp.x}) \cdots \right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ \left. + \chi_{sp}\right] & \\ 1 & \text{if } \frac{\left[(\rho_{px.sp} - \chi_{sp}) \cdot \xi_{sp.x} \cdot (2 - \xi_{sp.x}) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34$ $\lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp} \cdot (\lambda_p - \lambda_{p0.sp}) + \lambda_p\right] = 0.782$

$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^{2} - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 14.256 \cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z} \cdot \sigma_{E.sp.z} = 57.024 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \frac{\pi^2 \cdot E \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 180.432 \cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) \ldots\right] & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) \ldots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{sp} & \\ 1 \text{ if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.891$

Top plate stiffeners - check

x-direction

$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \leq 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} \text{ if } \lambda_p > 0.748 \end{cases} = 1$

$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$

$\sigma_{E.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^2}{12 \cdot (1 - \nu^2) \cdot h_{so}^2} = 2.479 \times 10^3 \cdot MPa$

$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 \\ \frac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{tps.x} = 0 \\ 7.81 - 6.29 \cdot \psi_{tps.x} + 9.78 \cdot \psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{tps.x} = -1 \\ 5.98 \cdot \left(1 - \psi_{tps.x}\right)^2 \text{ if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 \text{ if } \psi_{tps.x} < -3 \end{cases} = 4.117$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x} \cdot \sigma_{E.tps.x} = 1.021 \times 10^4 \cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \frac{\pi^2 \cdot E \cdot t_{so}^2}{12 \cdot (1 - \nu^2) \cdot s_{ts}^2} = 25.344 \cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \ldots\right] & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \ldots\right] \cdot \alpha_{ult}}{\gamma_{M1}} < 1 \\ + \chi_{tps} & \\ 1 \text{ if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right] \cdot \alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1$

Top plate check

$\left(\frac{\sigma_{x.Ed.tp}}{\rho_{c.tp} \cdot f_y(t_d)}{\gamma_{M1}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tp}}{\chi_w \cdot f_y(t_d)}{\gamma_{M1}}\right)^2 = 0.991$

$3 \left(\frac{\tau_{Ed.tp}}{\chi_w \cdot f_y(t_d)}{\gamma_{M1}}\right)^2 = 4.106 \times 10^{-3}$

$\left(\frac{\sigma_{x.Ed.tp}}{\rho_{c.tp} \cdot f_y(t_d)}{\gamma_{M1}}\right)^2 = 0.987$

$c_{tp} := \min\left(1.25 - 0.25 \cdot \psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp} \cdot \left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right) \cdot \xi_{tp} \cdot \left(2 - \xi_{tp}\right) + \chi_{tp} = 0.758$

$e_3 := \max\left(1.25 + 0.75 \cdot \chi_w^2, 2\right) = 2.33$

$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\rho_{c.tp.V} \cdot f_y(t_d)}{\gamma_{M1}}\right)^2 + \left(\frac{\sqrt{3} \cdot \tau_{Ed.tp}}{\chi_w \cdot f_y(t_d)}{\gamma_{M1}}\right)^{e_3}} = 0.994$

Side plate check

$\left(\frac{\sigma_{x.Ed.sp}}{\rho_{c.sp.x} \cdot f_y(t_s)}{\gamma_{M1}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\rho_{c.sp.z} \cdot f_y(t_s)}{\gamma_{M1}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\rho_{c.sp.x} \cdot f_y(t_s)}{\gamma_{M1}}\right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\rho_{c.sp.z} \cdot f_y(t_s)}{\gamma_{M1}}\right) \ldots = 0.604$
$+ 3 \cdot \left(\frac{\tau_{Ed.sp}}{\chi_w \cdot f_y(t_s)}{\gamma_{M1}}\right)^2$

With the V-factor

$c_{sp} := \min\left(1.25 - 0.25 \cdot \psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp} \cdot \left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right) \cdot \xi_{sp.x} \cdot \left(2 - \xi_{sp.x}\right) + \chi_{sp} = 1$

$e_{3w} = 1.25 + 0.75 \cdot \chi_w^2 = 2.33$

$V_{sp} := \rho_{c.sp.x.V} \cdot \rho_{c.sp.z} = 0.891$

$\xi_{sp} := \lambda_p - 0.5 \cdot \ln\left(\xi_{sp.z} + 1\right)^{0.9} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right) \cdot \xi_{sp} = 0.891$

$\sqrt{\left(\frac{\sigma_{x.Ed.sp}}{\rho_{c.sp.x.V} \cdot f_y(t_s)}{\gamma_{M1}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\rho_{c.sp.z.V} \cdot f_y(t_s)}{\gamma_{M1}}\right)^2 - V_{sp} \cdot \left(\frac{\sigma_{x.Ed.sp}}{\rho_{c.sp.x.V} \cdot f_y(t_s)}{\gamma_{M1}}\right) \cdot \left(\frac{\sigma_{z.Ed.sp}}{\rho_{c.sp.z.V} \cdot f_y(t_s)}{\gamma_{M1}}\right) \ldots} = 0.762$
$+ \left(\frac{\sqrt{3} \cdot \tau_{Ed.sp}}{\chi_w \cdot f_y(t_s)}{\gamma_{M1}}\right)^{e_3}$

Top plate stiffeners check

$\left(\frac{\sigma_{x.Ed.tps}}{\rho_{c.tps.x} \cdot f_y(t_s)}{\gamma_{M1}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tps}}{\chi_w \cdot f_y(t_s)}{\gamma_{M1}}\right)^2 = 0.556$

$c_{tps} := \min\left(1.25 - 0.25 \cdot \psi_{tps.x}, 1.25\right) = 1.099$

$\rho_{tps.x.V} := \min\left[c_{tps} \cdot \left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V} - \chi_{tps}\right) \cdot \xi_{tps.x} \cdot \left(2 - \xi_{tps.x}\right) + \chi_{tps} = 1$

$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\rho_{c.tps.x.V} \cdot f_y(t_s)}{\gamma_{M1}}\right)^2 + 3 \cdot \left(\frac{\tau_{Ed.tps}}{\chi_w \cdot f_y(t_s)}{\gamma_{M1}}\right)^{e_3}} = 0.744$

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\,MPa & \text{if } t \le 40\,mm \\ (335\,MPa) & \text{if } t > 40\,mm \end{cases}$ $\gamma_{M1} := 1$

$E := 210\,GPa$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\,GPa$

Geometry

Bottom plate

$t_b := 16\,mm$ $b_b := 1000\,mm$ $A_b := b_b \cdot t_b = 0.016\,m^2$

Top plate

$t_d := 10\,mm$ $b_d := 3500\,mm$ $A_d := b_d \cdot t_d = 0.035\,m^2$

Side plates

$t_s := 12\,mm$ $h_s := 389.2\,mm$ $A_s := h_s \cdot t_s = 4.67\times10^{-3}\,m^2$

Webs

$t_w := 12\,mm$ $h_w := 1340\,mm$ $A_w := h_w \cdot t_w = 0.016\,m^2$

$h_{w.x} := 1250\,mm$ $h_{w.y} := 482.8\,mm$

Top stiffeners

$t_{so} := 16\,mm$ $h_{so} := 140\,mm$ $A_{so} := h_{so} \cdot t_{so} = 2.24\times10^{-3}\,m^2$

$s_{so} := 350\,mm$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\,mm$ $h_{su} := 120\,mm$ $A_{su} := h_{su} \cdot t_{su} = 1.44\times10^{-3}\,m^2$

$s_{su} := 440\,mm$ $n_{su} := 3$

Whole bridge $s_{ts} := 2\,m$

$L_{bridge} := 18\,m$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\,m$

$I_{tot} := 0.0148\,m^4$ From grasshopper

$Y_{tp} := 0.526956\,m$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 78\,\frac{kN}{m^2}$

$Q := q \cdot b_d = 273\,\frac{kN}{m}$

$M_{Ed} := \frac{Q \cdot L_{bridge}^2}{8} = 1.106\times10^4\,kN\cdot m$

$V_{Ed} := \frac{Q \cdot L_{bridge}}{2} = 2.457\times10^3\,kN$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}} \cdot \left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 267.481\,MPa$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d \cdot b_d}{n_{so}+2} \cdot \left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\,m^3$

$\tau_{Ed.tp} := \frac{V_{Ed} \cdot S_{tp}}{I_{tot} \cdot \min(t_s, t_{so})} = 15.761\,MPa$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - Y_{tp}) = 263.745\,MPa$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 11.375\,MPa$

$S_{sp} := (h_{ts} - t_d - Y_{tp}) \cdot t_s \cdot \frac{(h_{ts} - t_d - Y_{tp})}{2} + \frac{t_d \cdot b_d}{2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\,m^3$
$+ \frac{n_{so}}{2} \cdot h_{so} \cdot t_{so} \cdot \left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5 V_{Ed} \cdot S_{sp}}{I_{tot} \cdot t_s} = 68.25\,MPa$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}} \cdot (h_{ts} - t_d - Y_{tp}) = 263.745\,MPa$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\,m^3$
$+ \frac{t_d \cdot b_d}{n_{so}+2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed} \cdot S_{tps}}{I_{tot} \cdot t_{so}} = 15.348\,MPa$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.45\,\frac{MPa}{\frac{kN}{m^2}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_V := \sigma_{V.UNIT} \cdot q = 347.1\,MPa$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.023$

$\alpha_{cr.UNIT} := 227.33\,\frac{kN}{m^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.914$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.592$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2 \cdot E \cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\,MPa$

$I_{p.tp} := \frac{b_d \cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-7}\,m^4$

$y_{tp.sl.tp} := \frac{A_d\frac{t_d}{2} + n_{so}\cdot A_{so}\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}A_{so}} = 32.411\,mm$

$I_{sl.tp} := \frac{b_d \cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\,mm^4$
$+ n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.571$ $\sqrt[4]{\gamma_{tp}} = 4.256$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\cdot\left[\left(1+\alpha_{tp}^{\,2}\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}^{\,2}\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 639.127$$

$$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 990.256\cdot MPa$$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

$$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$$

$$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\cdot b_{sl.1.tp}+A_{so}} = 0.034\,m$$

Combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^{\,3}}{12} + b_{sl.1.tp}\cdot t_d\cdot\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \frac{t_{so}\cdot h_{so}^{\,3}}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}^{\,2}} = 1.026\times 10^3\cdot MPa$$

Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,m$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp}-t_d,\, t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right) = 0.046\,m$$

$$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5\left[1+\alpha_{e.tp}\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 0.79$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}^{\,2}-\lambda_p^{\,2}}},\,1\right) = 0.762$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}}-1,\,0\right),\,1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right)+\chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.762$$

Shear

$$\eta := 1.2 \qquad 7.1\ (2)$$

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$$

$$\frac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.78$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{(Y_{tp})}\cdot z+\sigma_{max}\right]_{-1}$$

$$\psi_{sp.x} := \frac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = -0.102$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts}-t_d-Y_{tp}\right) = 263.745\cdot MPa$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts}-t_d-h_s-Y_{tp}\right) = -27.011\cdot MPa$$

$$\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$$

$$\psi_{sp3.x} := \max\left(3+\psi_{sp.x},\,0\right) = 2.898$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p-0.055\cdot(\psi_{sp3.x})}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^2\right)\cdot h_s^{\,2}} = 180.432\cdot MPa$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \dfrac{8.2}{(1.05+\psi_{sp.x})} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81-6.29\cdot\psi_{sp.x}+9.78\cdot\psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.557$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.544\times 10^3\cdot MPa$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^{\,2}} = 6.833\cdot MPa$$

$$\alpha_{sp} := 0.21$$

$$\phi_{sp} := 0.5\left[1+\alpha_{sp}\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 0.717$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp}+\sqrt{\phi_{sp}^{\,2}-\lambda_p^{\,2}}},\,1\right) = 0.893$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}}-1,\,0\right),\,1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)\cdots\right] & \text{if } \frac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)\cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)+\chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.778$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$\sigma_{E.sp.z} := \dfrac{\pi^2\cdot E\cdot t_s^2}{12\left(1-\nu^2\right)\cdot s_{ts}^2} = 6.833\cdot MPa$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 27.331\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.sp.z} := \dfrac{\pi^2\cdot E\cdot t_s^2}{12\left(1-\nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$

Interpolation

$\xi_{sp.z} := \min\left(\max\left(\dfrac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) \ldots\right]\cdot\alpha_{ult} + \chi_{sp} & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) \ldots\right]\cdot\alpha_{ult} + \chi_{sp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.893$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$\psi_{tps.x} := \dfrac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$

$\sigma_{E.tps.x} := \dfrac{\pi^2\cdot E\cdot t_{so}^2}{12\left(1-\nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$

$$k_{\sigma.tps.x} := \begin{cases} 1 & \text{if } \psi_{tps.x} = 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{tps.x} < -3 \end{cases} = 4.117$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.021\times 10^4\cdot MPa$

Collumn-like behaviour

$\sigma_{cr.c.tps.x} := \dfrac{\pi^2\cdot E\cdot t_{so}^2}{12\left(1-\nu^2\right)\cdot s_{ts}^2} = 12.147\cdot MPa$

$\chi_{tps} := \chi_{sp}$

Interpolation

$\xi_{tps.x} := \min\left(\max\left(\dfrac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \ldots\right]\cdot\alpha_{ult} + \chi_{tps} & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) \ldots\right]\cdot\alpha_{ult} + \chi_{tps}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + 3\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.981$$

$$3\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 4.106\times 10^{-3}$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.977$$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\xi_{tp}\cdot\left(2 - \xi_{tp}\right) + \chi_{tp} = 0.762$

$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.33$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp.V}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^{e_3}} = 0.989$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y(t_s)}{\gamma_{M1}}}\right) \ldots = 0.604$$
$$+ 3\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2$$

With the V-factor

$$\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2\cdot 3 = 0.077$$

$c_{sp} := \min\left(1.25 - 0.25\cdot\psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp}\cdot\left(\dfrac{1}{\lambda_p} - \dfrac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right)\cdot\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) + \chi_{sp} = 1$

$e_3 = 1.25 + 0.75\cdot\chi_w^2 = 2.33$

$V_{sp} := \rho_{c.sp.x.V}\cdot\rho_{c.sp.z} = 0.893$

$\xi_{sp} := \lambda_p^{-0.5\cdot\ln\left(\xi_{sp.z} + 1\right)^{0.9}} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right)\cdot\xi_{sp} = 0.893$

$$\sqrt{\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 - V_{sp}\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right) \ldots = 0.762}$$
$$+ \left(\frac{\sqrt{3}\cdot\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.556$$

$$\varepsilon_{tps} := \min\left(1.25 - 0.25\cdot\psi_{tps.x}, 1.25\right) = 1.099$$

$$\rho_{tps.x.V} := \min\left[\varepsilon_{tps}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$$

$$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V} - \chi_{tps}\right)\cdot\xi_{tps.x}\left(2 - \xi_{tps.x}\right) + \chi_{tps} = 1$$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}} = 0.744$$

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b \cdot t_b = 0.016\text{m}^2$

Top plate

$t_d := 10\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d \cdot t_d = 0.035\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s \cdot t_s = 4.67\times10^{-3}\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w \cdot t_w = 0.016\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so} \cdot t_{so} = 2.24\times10^{-3}\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su} \cdot t_{su} = 1.44\times10^{-3}\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\text{m}$

$s_{ts} := 2.25\text{m}$

$I_{tot} := 0.0148\text{m}^4$ From grasshopper

$Y_{tp} := 0.526956\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 79\cdot\frac{\text{kN}}{\text{m}^2}$

$Q := q \cdot b_d = 276.5\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.12\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.489\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 270.91\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d \cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 15.963\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 267.127\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 11.521\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5 V_{Ed}\cdot S_{sp}}{I_{tot}\cdot t_s} = 69.125\cdot\text{MPa}$

Top plate stifeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 267.127\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 15.545\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.5\cdot\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

$\sigma_V := \sigma_{V.UNIT}\cdot q = 355.5\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 0.999$

$\alpha_{cr.UNIT} := 229.26\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.902$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.587$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-7}\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\cdot A_{so}} = 32.411\cdot\text{mm}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\cdot\text{mm}^4$
$+ n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.643$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\left[\left(1+\alpha_{tp}{}^2\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}{}^2\cdot(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{(\psi_{tp}+1)\cdot(1+\delta_{tp})} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 505.354$$

$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 782.99\cdot MPa$

Column-like behaviour [4.5.3]

$b_{1.inf.tp} := \dfrac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$

$b_{2.sup.tp} := \dfrac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$ Parts of the top plate that contributes to the stiffener

$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$

$y_{tp.sl.1.tp} := \dfrac{t_d\, b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\, b_{sl.1.tp} + A_{so}} = 0.034\,m$ combined center of gravity for stiffener and contributing top plate.

$I_{sl.1.tp} := \dfrac{b_{sl.1.tp}\cdot t_d{}^3}{12} + b_{sl.1.tp}\cdot t_d\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \dfrac{t_{so}\cdot h_{so}{}^3}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$ Combined moment of inertia

$\sigma_{cr.sl.tp} := \dfrac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}{}^2} = 811.02\cdot MPa$ Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$i_{tp} := \sqrt{\dfrac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,m$

$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d, t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.046\,m$

$\alpha_{e.tp} := 0.49 + \dfrac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$ Stiffener with open cross section (curve c)

$\phi_{tp} := 0.5\left[1+\alpha_{e.tp}\cdot(\lambda_p-0.2)+\lambda_p{}^2\right] = 0.785$ [6.3.1.2]

$\chi_{tp} := \min\left(\dfrac{1}{\phi_{tp}+\sqrt{\phi_{tp}{}^2-\lambda_p{}^2}}, 1\right) = 0.766$

Interpolation

$\xi_{tp} := \min\left(\max\left(\dfrac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$

$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot(2-\xi_{tp})+\chi_{tp}\right] & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot(2-\xi_{tp})+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.766$

Shear

$\eta := 1.2$ 7.1 (2)

$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \dfrac{0.83}{\eta} \\ \dfrac{0.83}{\lambda_p} & \text{if } \dfrac{0.83}{\eta} \le \lambda_p < 1.08 \\ \dfrac{1.37}{(0.7+\lambda_p)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$

$\dfrac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.765$

Side plates check

x-direction

$\sigma_{max} := f_y(t_d)$

$\sigma(z) := \left[\dfrac{-\sigma_{max}}{(Y_{tp})}\cdot z + \sigma_{max}\right] - 1$

$\psi_{sp.x} := \dfrac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = -0.102$

$\sigma_{x.top} := \dfrac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-Y_{tp}\right) = 267.127\cdot MPa$

$\sigma_{x.bot} := \dfrac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-h_s-Y_{tp}\right) = -27.357\cdot MPa$

$\psi_{sp3.x} := \max\left(3+\psi_{sp.x}, 0\right) = 2.898$ $\psi_{sp1} := \dfrac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$

$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p - 0.055\left(\psi_{sp3.x}\right)}{\lambda_p{}^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour

$\sigma_{E.sp.x} := \dfrac{\pi^2\cdot E\cdot t_s{}^2}{12\left(1-\nu^2\right)\cdot h_s{}^2} = 180.432\cdot MPa$

$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \dfrac{8.2}{\left(1.05+\psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29\cdot\psi_{sp.x} + 9.78\cdot\psi_{sp.x}{}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.557$

$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\, k_{\sigma.sp.x} = 1.544\times 10^3\cdot MPa$

Column-like behaviour

$\sigma_{cr.c.sp.x} := \dfrac{\pi^2\cdot E\cdot t_s{}^2}{12\left(1-\nu^2\right)\cdot s_{ts}{}^2} = 5.399\cdot MPa$

$\alpha_{sp} := 0.21$

$\phi_{sp} := 0.5\left[1+\alpha_{sp}\cdot\left(\lambda_p-0.2\right)+\lambda_p{}^2\right] = 0.713$

$\chi_{sp} := \min\left(\dfrac{1}{\phi_{sp}+\sqrt{\phi_{sp}{}^2-\lambda_p{}^2}}, 1\right) = 0.895$

Interpolation

$\xi_{sp.x} := \min\left(\max\left(\dfrac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$

$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)\cdots\right] & \text{if } \dfrac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)\cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)+\chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.774$

$$\rho_{pz.sp} := \begin{cases} 1 \text{ if } \lambda_p \le 0.673 \\ \dfrac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} \text{ if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_{ts}^2} = 5.399\cdot MPa$$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 21.595\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$$

Interpolation

$$\varepsilon_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) \ldots\right] + \chi_{sp} \text{ if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) \ldots\right] + \chi_{sp}}{\gamma_{M1}} < 1 \\ 1 \text{ if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.895$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 \text{ if } \lambda_p \le 0.748 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^2} \text{ if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$$

$$\sigma_{E.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$$

$$k_{\sigma.tps.x} := \begin{cases} 1 \text{ if } \psi_{tps.x} = 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} \text{ if } 1 > \psi_{sp.x} > 0 \\ 7.81 \text{ if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 \text{ if } 0 > \psi_{sp.x} > -1 \\ 23.9 \text{ if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 \text{ if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 \text{ if } \psi_{tps.x} < -3 \end{cases} = 4.117$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.021\times 10^4\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 9.598\cdot MPa$$

$\chi_{tps} := \chi_{sp}$

Interpolation

$$\varepsilon_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) \ldots\right] + \chi_{tps} \text{ if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) \ldots\right] + \chi_{tps}}{\gamma_{M1}} < 1 \\ 1 \text{ if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.997$$

$$3\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 4.212\times 10^{-3}$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.993$$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\varepsilon_{tp}\cdot\left(2 - \varepsilon_{tp}\right) + \chi_{tp} = 0.766$

$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.33$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp.V}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^{e_3}} = 0.997$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \ldots = 0.619$$
$$+ 3\cdot\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

With the V-factor

$c_{sp} := \min\left(1.25 - 0.25\cdot\psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right)\cdot\varepsilon_{sp.x}\cdot\left(2 - \varepsilon_{sp.x}\right) + \chi_{sp} = 1$

$e_{3V} = 1.25 + 0.75\cdot\chi_w^2 = 2.33$

$V_{sp} := \rho_{c.sp.x.V}\cdot\rho_{c.sp.z} = 0.895$

$f_{sp} := \lambda_p^{-0.5\cdot\ln\left(\varepsilon_{sp.z} + 1\right)^{0.9}} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right)\cdot f_{sp} = 0.895$

$$\sqrt{\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - V_{sp}\cdot\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \ldots + \left(\frac{\sqrt{3}\cdot\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^{e_3}} = 0.771$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.57$$

$$\varepsilon_{tps} := \min\left(1.25 - 0.25\cdot\psi_{tps.x}, 1.25\right) = 1.099$$

$$\rho_{tps.x.V} := \min\left[\varepsilon_{tps}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$$

$$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V} - \chi_{tps}\right)\cdot\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) + \chi_{tps} = 1$$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}} = 0.753$$

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\,\text{MPa} & \text{if } t \le 40\,\text{mm} \\ (335\,\text{MPa}) & \text{if } t > 40\,\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\,\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\,\text{mm}$ $b_b := 1000\,\text{mm}$ $A_b := b_b \cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 10\,\text{mm}$ $b_d := 3500\,\text{mm}$ $A_d := b_d \cdot t_d = 0.035\,\text{m}^2$

Side plates

$t_s := 12\,\text{mm}$ $h_s := 389.2\,\text{mm}$ $A_s := h_s \cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\,\text{mm}$ $h_w := 1340\,\text{mm}$ $A_w := h_w \cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\,\text{mm}$ $h_{w.y} := 482.8\,\text{mm}$

Top stiffeners

$t_{so} := 16\,\text{mm}$ $h_{so} := 140\,\text{mm}$ $A_{so} := h_{so} \cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\,\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\,\text{mm}$ $h_{su} := 120\,\text{mm}$ $A_{su} := h_{su} \cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\,\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\,\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\,\text{m}$ $s_{ts} := \frac{18}{7}\,\text{m} = 2.571\,\text{m}$

$I_{tot} := 0.0148\,\text{m}^4$ From grasshopper

$Y_{tp} := 0.526956\,\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 78\,\frac{\text{kN}}{\text{m}^2}$

$Q := q \cdot b_d = 273\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.106\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.457\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 267.481\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 15.761\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 11.375\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\cdot\text{m}^3$
$\quad + \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\cdot t_s} = 68.25\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 263.745\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\cdot\text{m}^3$
$\quad + \frac{t_d\cdot b_d}{n_{so}+2}\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 15.348\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.57\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_V := \sigma_{V.UNIT}\cdot q = 356.46\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 0.996$

$\alpha_{cr.UNIT} := 214.1\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.745$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.602$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 1$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-7}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\cdot A_{so}} = 32.411\cdot\text{mm}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\cdot\text{mm}^4$
$\quad + n_{so}\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.735$ $\sqrt[4]{\gamma_{tp}} = 4.256$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\left[\left(1+\alpha_{tp}^{\,2}\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}^{\,2}\cdot\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 387.351$$

$$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 600.157\cdot\text{MPa}$$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,\text{m}$$

$$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,\text{m}$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,\text{m}$$

$$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)^{-3}\cdot 2}{t_d\cdot b_{sl.1.tp} + A_{so}} = 0.034\,\text{m}$$

Combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^{\,3}}{12} + b_{sl.1.tp}\cdot t_d\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \frac{t_{so}\cdot h_{so}^{\,3}}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}^{\,2}} = 620.937\cdot\text{MPa}$$

Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,\text{m}$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d, t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.046\,\text{m}$$

$$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$$

Stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5\left[1+\alpha_{e.tp}\cdot\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 0.799$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}^{\,2}-\lambda_p^{\,2}}}, 1\right) = 0.756$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}}-1, 0\right), 1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right] & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\left(2-\xi_{tp}\right)\cdots\right]\cdot\alpha_{ult}}{+\chi_{tp}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.756$$

Shear

$$\eta := 1.2$$

7.1 (2)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \dfrac{0.83}{\eta} \\ \dfrac{0.83}{\lambda_p} & \text{if } \dfrac{0.83}{\eta} \le \lambda_p < 1.08 \\ \dfrac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$$

$$\frac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.753$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}\cdot z + \sigma_{max}\right] - 1$$

$$\psi_{sp.x} := \frac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = -0.102$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts}-t_d-Y_{tp}\right) = 263.745\cdot\text{MPa}$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\left(h_{ts}-t_d-h_s-Y_{tp}\right) = -27.011\cdot\text{MPa}$$

$$\psi_{sp3.x} := \max\left(3+\psi_{sp.x}, 0\right) = 2.898 \qquad \psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p - 0.055\cdot\left(\psi_{sp3.x}\right)}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\left(1-\nu^2\right)\cdot h_s^{\,2}} = 180.432\cdot\text{MPa}$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \dfrac{8.2}{\left(1.05+\psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29\cdot\psi_{sp.x} + 9.78\cdot\psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.557$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.544\times 10^3\cdot\text{MPa}$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^{\,2}} = 4.133\cdot\text{MPa}$$

$$\alpha_{sp} := 0.21$$

$$\phi_{sp} := 0.5\left[1+\alpha_{sp}\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 0.724$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp}+\sqrt{\phi_{sp}^{\,2}-\lambda_p^{\,2}}}, 1\right) = 0.889$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}}-1, 0\right), 1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)\cdots\right] & \text{if } \dfrac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)\cdots\right]\cdot\alpha_{ult}}{+\chi_{sp}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\left(2-\xi_{sp.x}\right)+\chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.785$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 4.133\cdot MPa$$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 16.534\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$$

Interpolation

$$\varepsilon_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) \ldots\right] + \chi_{sp} & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) \ldots\right] + \chi_{sp}\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.889$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$$

$$\sigma_{E.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$$

$$k_{\sigma.tps.x} := \begin{cases} 1 & \text{if } \psi_{tps.x} = 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{tps.x} < -3 \end{cases} = 4.117$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.021\times 10^4\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 7.348\cdot MPa$$

$\chi_{tps} := \chi_{sp}$

Interpolation

$$\varepsilon_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) \ldots\right] + \chi_{tps} & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) \ldots\right] + \chi_{tps}\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.998$$

$$3\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 4.106\times 10^{-3}$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.994$$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\varepsilon_{tp}\cdot\left(2 - \varepsilon_{tp}\right) + \chi_{tp} = 0.756$

$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.33$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp.V}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^{e_3}} = 0.998$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \ldots + 3\cdot\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 = 0.603$$

With the V-factor

$c_{sp} := \min\left(1.25 - 0.25\cdot\psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right)\cdot\varepsilon_{sp.x}\cdot\left(2 - \varepsilon_{sp.x}\right) + \chi_{sp} = 1$

$e_3 := 1.25 + 0.75\cdot\chi_w^2 = 2.33$

$\chi_{sp.V} := \rho_{c.sp.x.V}\cdot\rho_{c.sp.z} = 0.889$

$f_{sp} := \lambda_p^{-0.5}\cdot\ln\left(\varepsilon_{sp.z} + 1\right)^{0.9} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right)\cdot f_{sp} = 0.889$

$$\sqrt{\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2+\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2-V_{sp}\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)+\left(\frac{\frac{\sqrt{3}\cdot\tau_{Ed.sp}}{\chi_w\cdot f_y(t_s)}}{\gamma_{M1}}\right)^{e_3}}\ldots = 0.762$$

Top plate stifeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2+3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.556$$

$$c_{tps} := \min\left(1.25 - 0.25\cdot\psi_{tps.x}, 1.25\right) = 1.099$$

$$\rho_{tps.x.V} := \min\left[c_{tps}\cdot\left(\frac{1}{\lambda_p}-\frac{0.22}{\lambda_p^2}\right), 1\right] = 1$$

$$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V} - \chi_{tps}\right)\cdot\varepsilon_{tps.x}\left(2-\varepsilon_{tps.x}\right)+\chi_{tps} = 1$$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2+3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}} = 0.744$$

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\text{m}^2$

Top plate

$t_d := 10\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.035\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\text{m}$

$I_{tot} := 0.0148\text{m}^4$ From grasshopper

$Y_{tp} := 0.526956\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 73\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 255.5\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.035\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.3\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 250.334\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 14.75\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 246.839\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 10.646\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\cdot t_s} = 63.875\cdot\text{MPa}$

Top plate stiffeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 246.839\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 14.365\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 4.66\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_V := \sigma_{V.UNIT}\cdot q = 340.18\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 1.044$

$\alpha_{cr.UNIT} := 166.25\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 2.277$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.677$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\left(3+\psi_{tp}\right)}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.997$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot\left(1-\nu^2\right)\cdot b_d^2} = 1.549\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot\left(1-\nu^2\right)} = 3.205\times10^{-7}\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\frac{t_d}{2} + n_{so}\cdot A_{so}\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}A_{so}} = 32.411\cdot\text{mm}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\cdot\text{mm}^4$
$+ n_{so}\cdot\left[\frac{t_{so}\cdot b_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 0.857$ $\sqrt[4]{\gamma_{tp}} = 4.256$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\cdot\left[\left(1+\alpha_{tp}^{\,2}\right)^{2}+\gamma_{tp}-1\right]}{\alpha_{tp}^{\,2}\cdot\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} \le \sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp} > \sqrt[4]{\gamma_{tp}} \end{cases} = 285.135$$

$$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 441.786\cdot MPa$$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

$$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\,m$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\,m$$

$$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\cdot b_{sl.1.tp} + A_{so}} = 0.034\,m$$

Combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d^{\,3}}{12} + b_{sl.1.tp}\cdot t_d\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \frac{t_{so}\cdot h_{so}^{\,3}}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}^{\,2}} = 456.199\cdot MPa$$

Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\,m$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp} - t_d - t_d + \frac{h_{so}}{2} - y_{tp.sl.1.tp}\right) = 0.046\,m$$

$$\alpha_{e.tp} := 0.49 + \frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5\left[1+\alpha_{e.tp}\cdot\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 0.868$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}^{\,2}-\lambda_p^{\,2}}}, 1\right) = 0.709$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right] & \text{if } \frac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.709$$

Shear

$$\eta := 1.2$$

7.1 (2)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta} \le \lambda_p < 1.08 \\ \frac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p \ge 1.08 \end{cases} = 1.2$$

$$\frac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.739$$

Side plates check

x-direction

$$\sigma_{max} := f_y\left(t_d\right)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}\cdot z + \sigma_{max}\right]\cdot 1$$

$$\psi_{sp.x} := \frac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = -0.102$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-Y_{tp}\right) = 246.839\cdot MPa$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-h_s-Y_{tp}\right) = -25.279\cdot MPa$$

$$\psi_{sp3.x} := \max\left(3+\psi_{sp.x}, 0\right) = 2.898 \qquad \psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{\lambda_p - 0.055\cdot\left(\psi_{sp3.x}\right)}{\lambda_p^{\,2}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.129$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^2\right)\cdot h_s^{\,2}} = 180.432\cdot MPa$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x} = 1 \\ \dfrac{8.2}{\left(1.05+\psi_{sp.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{sp.x} = 0 \\ 7.81 - 6.29\cdot\psi_{sp.x} + 9.78\cdot\psi_{sp.x}^{\,2} & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{sp.x} = -1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{sp.x} < -3 \end{cases} = 8.557$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.544\times 10^{3}\cdot MPa$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot t_s^{\,2}}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^{\,2}} = 3.037\cdot MPa$$

$$\alpha_{sp} := 0.21$$

$$\phi_{sp} := 0.5\left[1+\alpha_{sp}\cdot\left(\lambda_p-0.2\right)+\lambda_p^{\,2}\right] = 0.779$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp}+\sqrt{\phi_{sp}^{\,2}-\lambda_p^{\,2}}}, 1\right) = 0.858$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)\cdots\right] & \text{if } \frac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)\cdots\right]\cdot\alpha_{ult}}{\gamma_{M1}} < 1 \\ \left[+\chi_{sp}\right] & \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp}-\chi_{sp}\right)\xi_{sp.x}\cdot\left(2-\xi_{sp.x}\right)+\chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$\phi_{p.sp} := 0.5\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.835$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \dfrac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.026$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_{ts}^2} = 3.037\cdot MPa$$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 12.147\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1-\nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$$

Interpolation

$$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) \cdots\right] + \chi_{sp} & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) \cdots\right]\cdot\alpha_{ult} + \chi_{sp}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\varepsilon_{sp.z}\cdot\left(2 - \varepsilon_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 0.858$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \le 0.748 \\ \dfrac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$$

$$\sigma_{E.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$$

$$k_{\sigma.tps.x} := \begin{cases} 1 & \text{if } \psi_{tps.x} = 1 \\ \dfrac{8.2}{\left(1.05 + \psi_{tps.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \ge -3 \\ 95.68 & \text{if } \psi_{tps.x} < -3 \end{cases} = 4.117$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.021\times 10^4\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}^2} = 5.399\cdot MPa$$

$\chi_{tps} := \chi_{sp}$

Interpolation

$$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) \cdots\right] + \chi_{tps} & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) \cdots\right]\cdot\alpha_{ult} + \chi_{tps}}{\gamma_{M1}} < 1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \ge 1 \end{cases} = 1$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.994$$

$$3\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 3.597\times 10^{-3}$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 = 0.99$$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 0.997$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\varepsilon_{tp}\cdot\left(2 - \varepsilon_{tp}\right) + \chi_{tp} = 0.709$

$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.33$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp.V}\cdot f_y(t_d)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y(t_d)}{\gamma_{M1}}}\right)^{e_3}} = 0.996$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y(t_s)}{\gamma_{M1}}}\right) \cdots = 0.528$$
$$+ 3\cdot\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2$$

With the V-factor

$$\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2\cdot 3 = 0.067$$

$c_{sp} := \min\left(1.25 - 0.25\cdot\psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right)\cdot\varepsilon_{sp.x}\cdot\left(2 - \varepsilon_{sp.x}\right) + \chi_{sp} = 1$

$e_3 := 1.25 + 0.75\cdot\chi_w^2 = 2.33$

$V_{sp} := \rho_{c.sp.x.V}\cdot\rho_{c.sp.z} = 0.858$

$\xi_{sp} := \lambda_p^{-0.5}\cdot\ln\left(\varepsilon_{sp.z} + 1\right)^{0.9} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right)\cdot\xi_{sp} = 0.858$

$$\sqrt{\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 - V_{sp}\cdot\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)} \cdots = 0.712$$
$$+ \left(\frac{\sqrt{3}\cdot\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.487$$

$$\varepsilon_{tps} := \min\left(1.25 - 0.25\cdot\psi_{tps.x}, 1.25\right) = 1.099$$

$$\rho_{tps.x.V} := \min\left[\varepsilon_{tps}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$$

$$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V} - \chi_{tps}\right)\cdot\varepsilon_{tps.x}\cdot\left(2 - \varepsilon_{tps.x}\right) + \chi_{tps} = 1$$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}} = 0.696$$

The Reduced stress method

-Different vertical stiffener distance s_{ts}

Material

$f_y(t) := \begin{cases} 355\text{MPa} & \text{if } t \le 40\text{mm} \\ (335\text{MPa}) & \text{if } t > 40\text{mm} \end{cases}$ $\gamma_{M1} := 1$

$E := 210\text{GPa}$

$\nu := 0.3$ $G := \frac{E}{2\cdot(1+\nu)} = 80.769\cdot\text{GPa}$

Geometry

Bottom plate

$t_b := 16\text{mm}$ $b_b := 1000\text{mm}$ $A_b := b_b\cdot t_b = 0.016\,\text{m}^2$

Top plate

$t_d := 10\text{mm}$ $b_d := 3500\text{mm}$ $A_d := b_d\cdot t_d = 0.035\,\text{m}^2$

Side plates

$t_s := 12\text{mm}$ $h_s := 389.2\text{mm}$ $A_s := h_s\cdot t_s = 4.67\times10^{-3}\,\text{m}^2$

Webs

$t_w := 12\text{mm}$ $h_w := 1340\text{mm}$ $A_w := h_w\cdot t_w = 0.016\,\text{m}^2$

$h_{w.x} := 1250\text{mm}$ $h_{w.y} := 482.8\text{mm}$

Top stiffeners

$t_{so} := 16\text{mm}$ $h_{so} := 140\text{mm}$ $A_{so} := h_{so}\cdot t_{so} = 2.24\times10^{-3}\,\text{m}^2$

$s_{so} := 350\text{mm}$ $n_{so} := 9$

Web stiffeners

$t_{su} := 12\text{mm}$ $h_{su} := 120\text{mm}$ $A_{su} := h_{su}\cdot t_{su} = 1.44\times10^{-3}\,\text{m}^2$

$s_{su} := 440\text{mm}$ $n_{su} := 3$

Whole bridge

$L_{bridge} := 18\text{m}$ $h_{ts} := \frac{t_b}{2} + h_{w.y} + h_s + t_d = 0.89\,\text{m}$

$s_{ts} := 3.6\text{m}$

$I_{tot} := 0.0148\text{m}^4$ From grasshopper

$Y_{tp} := 0.526956\text{m}$ From grasshopper (from bottom of bottom plate)

Load

Global

$q := 72\,\frac{\text{kN}}{\text{m}^2}$

$Q := q\cdot b_d = 252\cdot\frac{\text{kN}}{\text{m}}$

$M_{Ed} := \frac{Q\cdot L_{bridge}^2}{8} = 1.021\times10^4\cdot\text{kN}\cdot\text{m}$

$V_{Ed} := \frac{Q\cdot L_{bridge}}{2} = 2.268\times10^3\cdot\text{kN}$

Top plate

$\sigma_{x.Ed.tp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 246.905\cdot\text{MPa}$ $\sigma_{z.Ed.tp} := 0$

$S_{tp} := \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) = 1.139\times10^{-3}\cdot\text{m}^3$

$\tau_{Ed.tp} := \frac{V_{Ed}\cdot S_{tp}}{I_{tot}\cdot\min(t_s, t_{so})} = 14.548\cdot\text{MPa}$

Side plates

$\sigma_{x.Ed.sp} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 243.457\cdot\text{MPa}$

$\sigma_{z.Ed.sp} := \frac{Q}{2t_s} = 10.5\cdot\text{MPa}$

$S_{sp} := \left(h_{ts} - t_d - Y_{tp}\right)\cdot t_s\cdot\frac{\left(h_{ts} - t_d - Y_{tp}\right)}{2} + \frac{t_d\cdot b_d}{2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right) \ldots = 9.867\times10^{-3}\cdot\text{m}^3$
$+ \frac{n_{so}}{2}\cdot h_{so}\cdot t_{so}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{2} - Y_{tp}\right)$

$\tau_{Ed.sp} := \frac{0.5V_{Ed}\cdot S_{sp}}{I_{tot}\,t_s} = 63\cdot\text{MPa}$

Top plate stifeners

$\sigma_{x.Ed.tps} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts} - t_d - Y_{tp}\right) = 243.457\cdot\text{MPa}$

$S_{tps} := \frac{\left(n_{so}\cdot\frac{t_{so}\cdot h_{so}}{2} + 2\cdot\frac{t_s\cdot h_{so}}{2}\right)}{n_{so}+2}\cdot\left(h_{ts} - t_d - \frac{h_{so}}{4} - Y_{tp}\right) \ldots = 1.479\times10^{-3}\cdot\text{m}^3$
$+ \frac{t_d\cdot b_d}{n_{so}+2}\cdot\left(h_{ts} - \frac{t_d}{2} - Y_{tp}\right)$

$\tau_{Ed.tps} := \frac{V_{Ed}\cdot S_{tps}}{I_{tot}\cdot t_{so}} = 14.168\cdot\text{MPa}$

Governing slenderness from FE-software

$\sigma_{V.UNIT} := 5.85\,\frac{\text{MPa}}{\frac{\text{kN}}{\text{m}^2}}$ From a linear stress analysis in Sofistik

4.2449

$\sigma_V := \sigma_{V.UNIT}\cdot q = 421.2\cdot\text{MPa}$

$\alpha_{ult} := \frac{f_y(t_d)}{\sigma_V} = 0.843$

$\alpha_{cr.UNIT} := 126.17\cdot\frac{\text{kN}}{\text{m}^2}$ From a buckling eigenvalue analysis in Sofistik

$\alpha_{cr} := \frac{\alpha_{cr.UNIT}}{q} = 1.752$

$\lambda_p := \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = 0.694$ Slenderness for all parts

Top plate check

$\psi_{tp} := 1$

$\rho_{px.tp} := \begin{cases} 1 & \text{if } \lambda_p \le 0.673 \\ \frac{\lambda_p - 0.055\cdot(3+\psi_{tp})}{\lambda_p^2} & \text{if } \lambda_p > 0.673 \end{cases} = 0.985$

Plate-like behaviour [A.1]

$\sigma_{E.tp} := \frac{\pi^2\cdot E\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 1.549\cdot\text{MPa}$

$I_{p.tp} := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^{-7}\,\text{m}^4$

$y_{tp.sl.tp} := \frac{A_d\cdot\frac{t_d}{2} + n_{so}\cdot A_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right)}{A_d + n_{so}\cdot A_{so}} = 32.411\cdot\text{mm}$

$I_{sl.tp} := \frac{b_d\cdot t_d^3}{12} + A_d\cdot\left(y_{tp.sl.tp} - \frac{t_d}{2}\right)^2 \ldots = 1.052\times10^8\cdot\text{mm}^4$
$+ n_{so}\cdot\left[\frac{t_{so}\cdot h_{so}^3}{12} + A_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.tp}\right)^2\right]$

$\gamma_{tp} := \frac{I_{sl.tp}}{I_{p.tp}} = 328.143$

$\delta_{tp} := \frac{n_{so}\cdot A_{so}}{A_d} = 0.576$

$\alpha_{tp} := \max\left(\frac{s_{ts}}{b_d}, 0.5\right) = 1.029$ $\sqrt[4]{\gamma_{tp}} = 4.256$

$$k_{\sigma.p.tp} := \begin{cases} \dfrac{2\cdot\left[\left(1+\alpha_{tp}{}^2\right)^2+\gamma_{tp}-1\right]}{\alpha_{tp}{}^2\cdot\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp}\le\sqrt[4]{\gamma_{tp}} \\ \dfrac{4\cdot\left(1+\sqrt{\gamma_{tp}}\right)}{\left(\psi_{tp}+1\right)\cdot\left(1+\delta_{tp}\right)} & \text{if } \alpha_{tp}>\sqrt[4]{\gamma_{tp}} \end{cases} = 198.746$$

$$\sigma_{cr.p.tp} := k_{\sigma.p.tp}\cdot\sigma_{E.tp} = 307.935\cdot MPa$$

Column-like behaviour [4.5.3]

$$b_{1.inf.tp} := \frac{3-\psi_{tp}}{5-\psi_{tp}}\cdot s_{so} = 0.175\ m$$

$$b_{2.sup.tp} := \frac{2}{5-\psi_{tp}}\cdot s_{so} = 0.175\ m$$

Parts of the top plate that contributes to the stiffener

$$b_{sl.1.tp} := b_{1.inf.tp} + b_{2.sup.tp} = 0.35\ m$$

$$y_{tp.sl.1.tp} := \frac{t_d\cdot b_{sl.1.tp}\cdot\frac{t_d}{2} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}\right)}{t_d\cdot b_{sl.1.tp}+A_{so}} = 0.034\ m$$

combined center of gravity for stiffener and contributing top plate.

$$I_{sl.1.tp} := \frac{b_{sl.1.tp}\cdot t_d{}^3}{12} + b_{sl.1.tp}\cdot t_d\cdot\left(y_{tp.sl.1.tp}-\frac{t_d}{2}\right)^2 + \frac{t_{so}\cdot h_{so}{}^3}{12} + A_{so}\cdot\left(t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right)^2$$

Combined moment of inertia

$$\sigma_{cr.sl.tp} := \frac{\pi^2\cdot E\cdot I_{sl.1.tp}}{A_{sl.1.tp}\cdot s_{ts}{}^2} = 316.805\cdot MPa$$

Critical stress for kolumnlike buckling of stiffener

The top plate and stiffeners are subjected to a constant stress distribution so $\sigma_{cr.c} = \sigma_{cr.sl}$

$\sigma_{cr.c.tp} := \sigma_{cr.sl.tp}$

Reduction factor for column-like behaviour

$$i_{tp} := \sqrt{\frac{I_{sl.1.tp}}{A_{sl.1.tp}}} = 0.045\ m$$

$$e_{tp} := \max\left(y_{tp.sl.1.tp}-t_d,\ t_d+\frac{h_{so}}{2}-y_{tp.sl.1.tp}\right) = 0.046\ m$$

$$\alpha_{e.tp} := 0.49+\frac{0.09}{\frac{i_{tp}}{e_{tp}}} = 0.582$$

stiffener with open cross section (curve c)

$$\phi_{tp} := 0.5\cdot\left[1+\alpha_{e.tp}\cdot\left(\lambda_p-0.2\right)+\lambda_p{}^2\right] = 0.884$$

[6.3.1.2]

$$\chi_{tp} := \min\left(\frac{1}{\phi_{tp}+\sqrt{\phi_{tp}{}^2-\lambda_p{}^2}},1\right) = 0.698$$

Interpolation

$$\xi_{tp} := \min\left(\max\left(\frac{\sigma_{cr.p.tp}}{\sigma_{cr.c.tp}}-1,0\right),1\right) = 0$$

$$\rho_{c.tp} := \begin{cases} \left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right] & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}}<1 \\ 1 & \text{if } \dfrac{\left[\left(\rho_{px.tp}-\chi_{tp}\right)\xi_{tp}\cdot\left(2-\xi_{tp}\right)+\chi_{tp}\right]\cdot\alpha_{ult}}{\gamma_{M1}}\ge 1 \end{cases} = 0.698$$

Shear

$\eta := 1.2$ 7.1 (2)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_p<\frac{0.83}{\eta} \\ \frac{0.83}{\lambda_p} & \text{if } \frac{0.83}{\eta}\le\lambda_p<1.08 \\ \frac{1.37}{\left(0.7+\lambda_p\right)} & \text{if } \lambda_p\ge 1.08 \end{cases} = 1.197$$

$$\frac{\rho_{c.tp}\cdot\alpha_{ult}}{\gamma_{M1}} = 0.588$$

Side plates check

x-direction

$$\sigma_{max} := f_y(t_d)$$

$$\sigma(z) := \left[\frac{-\sigma_{max}}{\left(Y_{tp}\right)}\cdot z+\sigma_{max}\right]^{-1}$$

$$\psi_{sp.x} := \frac{\sigma\left(h_{ts}-t_d-h_s\right)}{\sigma\left(h_{ts}-t_d\right)} = -0.102$$

$$\sigma_{x.top} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-Y_{tp}\right) = 243.457\cdot MPa$$

$$\sigma_{x.bot} := \frac{M_{Ed}}{I_{tot}}\cdot\left(h_{ts}-t_d-h_s-Y_{tp}\right) = -24.933\cdot MPa$$

$$\psi_{sp1} := \frac{\sigma_{x.bot}}{\sigma_{x.top}} = -0.102$$

$$\psi_{sp3.x} := \max\left(3+\psi_{sp.x},0\right) = 2.898$$

$$\rho_{px.sp} := \begin{cases} 1 & \text{if } \lambda_p\le 0.673 \\ \dfrac{\lambda_p-0.055\cdot\left(\psi_{sp3.x}\right)}{\lambda_p{}^2} & \text{if } \lambda_p>0.673 \end{cases} = 1.111$$

Plate-like behaviour

$$\sigma_{E.sp.x} := \frac{\pi^2\cdot E\cdot t_s{}^2}{12\cdot\left(1-\nu^2\right)\cdot h_s{}^2} = 180.432\cdot MPa$$

$$k_{\sigma.sp.x} := \begin{cases} 1 & \text{if } \psi_{sp.x}=1 \\ \dfrac{8.2}{\left(1.05+\psi_{sp.x}\right)} & \text{if } 1>\psi_{sp.x}>0 \\ 7.81 & \text{if } \psi_{sp.x}=0 \\ 7.81-6.29\cdot\psi_{sp.x}+9.78\cdot\psi_{sp.x}{}^2 & \text{if } 0>\psi_{sp.x}>-1 \\ 23.9 & \text{if } \psi_{sp.x}=-1 \\ 5.98\cdot\left(1-\psi_{sp.x}\right)^2 & \text{if } -1>\psi_{sp.x}\ge -3 \\ 95.68 & \text{if } \psi_{sp.x}<-3 \end{cases} = 8.557$$

$$\sigma_{cr.p.sp.x} := \sigma_{E.sp.x}\cdot k_{\sigma.sp.x} = 1.544\times 10^3\cdot MPa$$

Column-like behaviour

$$\sigma_{cr.c.sp.x} := \frac{\pi^2\cdot E\cdot t_s{}^2}{12\cdot\left(1-\nu^2\right)\cdot s_{ts}{}^2} = 2.109\cdot MPa$$

$\alpha_{sp} := 0.21$

$$\phi_{sp} := 0.5\cdot\left[1+\alpha_{sp}\cdot\left(\lambda_p-0.2\right)+\lambda_p{}^2\right] = 0.792$$

$$\chi_{sp} := \min\left(\frac{1}{\phi_{sp}+\sqrt{\phi_{sp}{}^2-\lambda_p{}^2}},1\right) = 0.851$$

Interpolation

$$\xi_{sp.x} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.x}}{\sigma_{cr.c.sp.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.sp.x} := \begin{cases} \left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) \cdots\right] + \chi_{sp} & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) \cdots\right] + \chi_{sp}}{\gamma_{M1}} \cdot \alpha_{ult} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.sp} - \chi_{sp}\right)\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 1.111$$

Z-direction

$\psi_{sp.z} := 1$

$\alpha_{p.sp} := 0.34 \qquad \lambda_{p0.sp} := 0.7$

$$\phi_{p.sp} := 0.5\cdot\left[1 + \alpha_{p.sp}\cdot\left(\lambda_p - \lambda_{p0.sp}\right) + \lambda_p\right] = 0.846$$

$$\rho_{pz.sp} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.673 \\ \frac{1}{\phi_{p.sp} + \sqrt{\phi_{p.sp}^2 - \lambda_p}} & \text{if } \lambda_p > 0.673 \end{cases} = 1.007$$

Plate-like behaviour

$k_{\sigma.p.sp.z} := 4$

$$\sigma_{E.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1 - \nu^2\right)\cdot s_{ts}^2} = 2.109\cdot MPa$$

$\sigma_{cr.p.sp.z} := k_{\sigma.p.sp.z}\cdot\sigma_{E.sp.z} = 8.436\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.sp.z} := \frac{\pi^2\cdot E\cdot t_s^2}{12\cdot\left(1 - \nu^2\right)\cdot h_s^2} = 180.432\cdot MPa$$

Interpolation

$$\xi_{sp.z} := \min\left(\max\left(\frac{\sigma_{cr.p.sp.z}}{\sigma_{cr.c.sp.z}} - 1, 0\right), 1\right) = 0$$

$$\rho_{c.sp.z} := \begin{cases} \left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] + \chi_{sp} & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) \cdots\right] + \chi_{sp}}{\gamma_{M1}} \cdot \alpha_{ult} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{pz.sp} - \chi_{sp}\right)\xi_{sp.z}\cdot\left(2 - \xi_{sp.z}\right) + \chi_{sp}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases} = 0.851$$

Top plate stiffeners - check

x-direction

$$\rho_{px.tps} := \begin{cases} 1 & \text{if } \lambda_p \leq 0.748 \\ \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \end{cases} = 1$$

$$\psi_{tps.x} := \frac{\sigma\left(h_{ts} - t_d - h_{so}\right)}{\sigma\left(h_{ts} - t_d\right)} = 0.603$$

$$\sigma_{E.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1 - \nu^2\right)\cdot h_{so}^2} = 2.479\times 10^3\cdot MPa$$

$$k_{\sigma.tps.x} := \begin{cases} 1 & \text{if } \psi_{tps.x} = 1 \\ \frac{8.2}{\left(1.05 + \psi_{tps.x}\right)} & \text{if } 1 > \psi_{sp.x} > 0 \\ 7.81 & \text{if } \psi_{tps.x} = 0 \\ 7.81 - 6.29\cdot\psi_{tps.x} + 9.78\cdot\psi_{sp.x}^2 & \text{if } 0 > \psi_{sp.x} > -1 \\ 23.9 & \text{if } \psi_{tps.x} = -1 \\ 5.98\cdot\left(1 - \psi_{tps.x}\right)^2 & \text{if } -1 > \psi_{sp.x} \geq -3 \\ 95.68 & \text{if } \psi_{tps.x} < -3 \end{cases} = 4.117$$

$\sigma_{cr.p.tps.x} := k_{\sigma.tps.x}\cdot\sigma_{E.tps.x} = 1.021\times 10^4\cdot MPa$

Collumn-like behaviour

$$\sigma_{cr.c.tps.x} := \frac{\pi^2\cdot E\cdot t_{so}^2}{12\cdot\left(1 - \nu^2\right)\cdot s_{ts}^2} = 3.749\cdot MPa$$

$\chi_{tps} := \chi_{sp}$

Interpolation

$$\xi_{tps.x} := \min\left(\max\left(\frac{\sigma_{cr.p.tps.x}}{\sigma_{cr.c.tps.x}} - 1, 0\right), 1\right) = 1$$

$$\rho_{c.tps.x} := \begin{cases} \left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] + \chi_{tps} & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) \cdots\right] + \chi_{tps}}{\gamma_{M1}} \cdot \alpha_{ult} < 1 \\ 1 & \text{if } \frac{\left[\left(\rho_{px.tps} - \chi_{tps}\right)\xi_{tps.x}\cdot\left(2 - \xi_{tps.x}\right) + \chi_{tps}\right]\cdot\alpha_{ult}}{\gamma_{M1}} \geq 1 \end{cases}$$

Top plate check

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + 3\cdot\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.996$$

$$3\left(\frac{\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 3.518\times 10^{-3}$$

$$\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 = 0.993$$

$c_{tp} := \min\left(1.25 - 0.25\cdot\psi_{tp}, 1.25\right) = 1$

$\rho_{tp.V} := \min\left[c_{tp}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 0.985$

$\rho_{c.tp.V} := \left(\rho_{tp.V} - \chi_{tp}\right)\cdot\xi_{tp}\cdot\left(2 - \xi_{tp}\right) + \chi_{tp} = 0.698$

$e_3 := \max\left(1.25 + 0.75\cdot\chi_w^2, 2\right) = 2.324$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tp}}{\frac{\rho_{c.tp.V}\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sqrt{3}\cdot\tau_{Ed.tp}}{\frac{\chi_w\cdot f_y\left(t_d\right)}{\gamma_{M1}}}\right)^{e_3}} = 0.997$$

Side plate check

$$\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z}\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right) \cdots = 0.427$$

$$+ 3\cdot\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2$$

With the V-factor

$$\left(\frac{\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y\left(t_s\right)}{\gamma_{M1}}}\right)^2\cdot 3 = 0.066$$

$c_{sp} := \min\left(1.25 - 0.25\cdot\psi_{sp.x}, 1.25\right) = 1.25$

$\rho_{sp.x.V} := \min\left[c_{sp}\cdot\left(\frac{1}{\lambda_p} - \frac{0.22}{\lambda_p^2}\right), 1\right] = 1$

$\rho_{c.sp.x.V} := \left(\rho_{sp.x.V} - \chi_{sp}\right)\cdot\xi_{sp.x}\cdot\left(2 - \xi_{sp.x}\right) + \chi_{sp} = 1$

$e_{3.V} := 1.25 + 0.75\cdot\chi_w^2 = 2.324$

$V_{sp} := \rho_{c.sp.x.V}\cdot\rho_{c.sp.z} = 0.851$

$f_{sp} := \lambda_p^{-0.5}\cdot\ln\left(\xi_{sp.z} + 1\right)^{0.9} = 0$

$\rho_{c.sp.z.V} := \chi_{sp} + \left(\rho_{pz.sp} - \chi_{sp}\right)\cdot f_{sp} = 0.851$

$$\sqrt{\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2+\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2-V_{sp}\cdot\left(\frac{\sigma_{x.Ed.sp}}{\frac{\rho_{c.sp.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)\cdot\left(\frac{\sigma_{z.Ed.sp}}{\frac{\rho_{c.sp.z.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)+\left(\frac{\sqrt{3}\cdot\tau_{Ed.sp}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}}\ \ldots = 0.703$$

Top plate stiffeners check

$$\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2+3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2 = 0.474$$

$c_{tps} := \min\left(1.25 - 0.25\cdot\psi_{tps.x}, 1.25\right) = 1.099$

$\rho_{tps.x.V} := \min\left[c_{tps}\cdot\left(\frac{1}{\lambda_p}-\frac{0.22}{\lambda_p^{\ 2}}\right), 1\right] = 1$

$\rho_{c.tps.x.V} := \left(\rho_{tps.x.V}-\chi_{tps}\right)\cdot\xi_{tps.x}\cdot\left(2-\xi_{tps.x}\right)+\chi_{tps} = 1$

$$\sqrt{\left(\frac{\sigma_{x.Ed.tps}}{\frac{\rho_{c.tps.x.V}\cdot f_y(t_s)}{\gamma_{M1}}}\right)^2+3\cdot\left(\frac{\tau_{Ed.tps}}{\frac{\chi_w\cdot f_y(t_s)}{\gamma_{M1}}}\right)^{e_3}} = 0.687$$

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 1.6m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 9.169 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Inre tvärsnittsdelar, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Höjd dragen del

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Höjd tryckt del

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_{a.v} := t_d = 10 \cdot mm$

$z_{a.v} := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2b}\left(\psi_{so}\right), h_{so}, t_{so}\right) = 0.484$

$\rho_{d.so} := \rho_{4.3}\left(\lambda_{p.so}, \psi_s\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m$ $\qquad \psi_d := \psi_{neg}\left(z_d, z_d\right) = 1$

$\delta_{so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}\left(\psi_d\right) = 4$

$\lambda_{p.d} := \lambda_p\left(k_{\sigma.d}, s_{c.so}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.31 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.053 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^{5} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \ldots + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.457$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 997.731$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot \left(1 - \nu^2\right) \cdot b_d^{2}} = 1.546 \times 10^{3} \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{cr.p} := \sqrt{\frac{\beta_{A.c} \cdot f_y\left(t_d\right)}{\sigma_{cr.p}}} = 0.467$

$\rho := \rho_{4.2}\left(\lambda_p, \psi\right) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, \psi\right) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 5.375 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 1.908 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \ldots + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{7} \cdot mm^4$

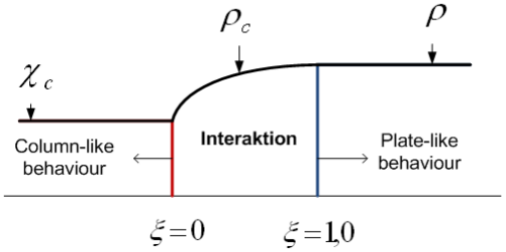
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.604 \times 10^{3} \cdot MPa$

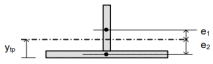
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.459$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.673$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.859$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.859$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -319.999 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 304.813 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 371.724 \cdot mm$

$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.788 \times 10^{-4}\,m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323 \times 10^{-3}\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -5.468 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 466.024 \cdot cm^2$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.127 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 8.586 \cdot mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 379.851 \cdot mm$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.517 \times 10^{-3}\,m^4$$

$$s_{su0} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.846 \times 10^{-3}\,m^4$$

$$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4 \quad \text{from FEM}$$

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 8.333 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4 \quad \text{from FEM}$$

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.811 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 4.035 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 4.048 \times 10^{-4}\,m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.445 \times 10^{10} \cdot mm^4$$

Load

$q_M := 95\,\frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$$Q := q_M \cdot b_d = 332.5 \cdot \frac{kN}{m} \qquad V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.993 \times 10^{3} \cdot kN$$

$$M_{Ed} := \frac{Q \cdot l_{bridge}^{2}}{8} = 1.347 \times 10^{4} \cdot kN \cdot m$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.038 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.35 \times 10^{4} \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.997$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119 \qquad \text{Angle of web}$$

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4 \qquad \text{(two stiffeners)}$$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15εt of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.577$$

SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.451 \qquad \text{EC1-3-5 eq.5.6}$$

Shear buckling coefficient; web with more than two longitudinal stiffeners

$$\alpha := \frac{a}{h_w} = 1.194$$

SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 7.471 \times 10^{3}$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 2.519 \times 10^{3}$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \le 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.451$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.993 \times 10^3 \cdot kN$

$$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.704$$

$$V_{b.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$$a_{\ddot{o}f.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 374.851 \cdot mm$$

$$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 502.149 \cdot mm$$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{\ddot{o}f.eff.neg} \cdot |a_{\ddot{o}f.eff.neg}| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot |a_{uf.eff.neg}| \cdot f_y(t_b)}{\gamma_{M0}} = 7.13 \times 10^3 \cdot kN \cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{\ddot{o}f} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.498\,m$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.472\,m$$

$$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{\ddot{o}f} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 368.345 \cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot kN$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.704$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.552 \times 10^{-3} \cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 452.924 \cdot cm^2$

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 1.8m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 9.169 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Inre tvärsnittsdelar, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Höjd dragen del

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Höjd tryckt del

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_w := t_d = 10 \cdot mm$

$z_2 := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2b}\left(\psi_{so}\right), h_{so}, t_{so}\right) = 0.484$

$\rho_{d.so} := \rho_{4.3}\left(\lambda_{p.so}, \psi_s\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^4 \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m$ $\qquad \psi_d := \psi_{neg}\left(z_d, z_d\right) = 1$

$\delta s_{so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}\left(\psi_d\right) = 4$

$\lambda_{p.d} := \lambda_p\left(k_{\sigma.d}, s_{c.so}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.31 \times 10^4 \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.053 \times 10^4 \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^5 \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^8 \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.514$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 788.66$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_d^2} = 1.222 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{p.p} := \sqrt{\frac{\beta_{A.c} \cdot f_y\left(t_d\right)}{\sigma_{cr.p}}} = 0.526$

$\rho := \rho_{4.2}\left(\lambda_p, \psi\right) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, \psi\right) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 5.375 \times 10^4 \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 1.908 \times 10^4 \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^3}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^7 \cdot mm^4$

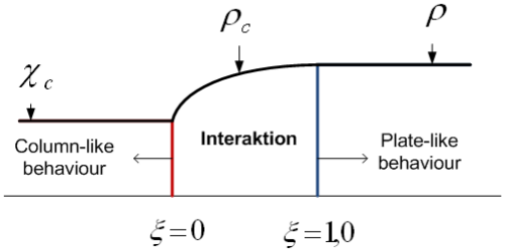
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^3 \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^3 \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.267 \times 10^3 \cdot MPa$

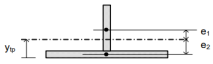
Slenderness ratio

$$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.516$$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$$

$$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$$

$$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$$

$$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^{\,2}\right] = 0.716$$ EC1993-1-1 6.3.1.2 (1)

$$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^{\,2}}}\right) = 0.826$$

Interaction

$$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.826$$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$$F_{avst} := n_{so}\, s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571 \times 10^4 \cdot kN$$

$$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so}\, s_{so.eff} \cdot t_d + A_{so}} = -319.999 \cdot MPa$$

Check according to A.2.1 (4)

$$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 293.087 \cdot MPa$$

$$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$$A_{d.eff} := n_{s.so}\, t_d \cdot s_{so.eff} = 0.032\,m^2$$

$$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 371.724 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^{\,3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^{\,3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^{\,3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.788 \times 10^{-4}\,m^4$$

$$I_{y.d.eff} := \frac{n_{s.so}\, s_{so.eff} \cdot t_d^{\,3}}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323 \times 10^{-3}\,m^4$$

$$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$$A_{öf.eff.neg} := A_{d.eff}$$

$$A_{c.eff.loc} = 5.053 \times 10^4 \cdot mm^2$$

$$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -5.257 \times 10^3 \cdot mm^2$$

$$A_{uf.eff.neg} := A_b$$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 449.332 \cdot cm^2$$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.111 \times 10^3 \cdot cm^2$$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 8.256 \cdot mm$$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 385.036 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.643 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.909 \times 10^{-3}\,m^4$$

$$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4 \quad \text{from FEM}$$

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 8.084 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4 \quad \text{from FEM}$$

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.724 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.952 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array} \right] = 4.22 \times 10^{-4}\,m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.446 \times 10^{10} \cdot mm^4$$

Load

$q_M := 94 \frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$$Q := q_M \cdot b_d = 329 \cdot \frac{kN}{m} \qquad V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.961 \times 10^3 \cdot kN$$

$$M_{Ed} := \frac{Q \cdot l_{bridge}^2}{8} = 1.332 \times 10^4 \cdot kN \cdot m$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.038 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.333 \times 10^4 \cdot kN \cdot m$$

$\frac{M_{Ed}}{M_{Rd}} = 1$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119 \qquad \text{Angle of web}$$

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4 \qquad \text{(two stiffeners)}$$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15εt of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.527$$

SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{tst} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.453 \qquad \text{EC1-3-5 eq.5.6}$$

Shear buckling coefficient; web with more than two longitudinal stiffeners

$$\alpha := \frac{a}{h_w} = 1.343$$

SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 5.903 \times 10^3$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 2.004 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \le 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.453$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.961 \times 10^3 \cdot kN$

$$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.697$$

$$V_{b.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 380.036 \cdot mm$$

$$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 496.964 \cdot mm$$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot |a_{öf.eff.neg}| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot |a_{uf.eff.neg}| \cdot f_y(t_b)}{\gamma_{M0}} = 7.16 \times 10^3 \cdot kN \cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.56\,m$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.531\,m$$

$$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 327.418 \cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot kN$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.697$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.595 \times 10^{-3} \cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 442.892 \cdot cm^2$

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} 355MPa & \text{if } t \le 40mm \\ 335MPa & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 9.169 \times 10^{-4}\,m^4$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Inre tvärsnittsdelar, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Höjd dragen del

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Höjd tryckt del

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_{a.w} := t_d = 10 \cdot mm$

$z_{a.f} := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2b}\left(\psi_{so}\right), h_{so}, t_{so}\right) = 0.484$

$\rho_{d.so} := \rho_{4.3}\left(\lambda_{p.so}, \psi_s\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^4 \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m$ $\psi_d := \psi_{neg}\left(z_d, z_d\right) = 1$

$\delta_{s.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}\left(\psi_d\right) = 4$

$\lambda_{p.d} := \lambda_p\left(k_{\sigma.d}, s_{c.so}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.31 \times 10^4 \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.053 \times 10^4 \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^5 \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^8 \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.571$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 639.127$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_d^2} = 990.256 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_p := \sqrt{\frac{\beta_{A.c} \cdot f_y\left(t_d\right)}{\sigma_{cr.p}}} = 0.584$

$\rho := \rho_{4.2}\left(\lambda_p, \psi\right) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, \psi\right) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 5.375 \times 10^4 \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 1.908 \times 10^4 \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^3}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^7 \cdot mm^4$

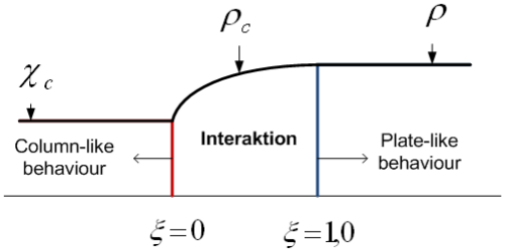
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^3 \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^3 \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 1.026 \times 10^3 \cdot MPa$

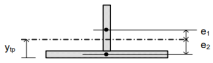
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.573$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.762$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.792$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.792$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -319.999 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 281.032 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 371.724 \cdot mm$

$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.788 \times 10^{-4}\,m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323 \times 10^{-3}\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -5.041 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{w.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 432.173 \cdot cm^2$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.094 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d \end{cases} = 7.916 \cdot mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 390.532 \cdot mm$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.778 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 1.977 \times 10^{-3}\,m^4$$

$$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4 \quad \text{from FEM}$$

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 7.825 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4 \quad \text{from FEM}$$

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.635 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.865 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 4.408 \times 10^{-4}\,m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.448 \times 10^{10} \cdot mm^4$$

Load

$q_M := 92\,\frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$$Q := q_M \cdot b_d = 322 \cdot \frac{kN}{m} \qquad V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.898 \times 10^{3} \cdot kN$$

$$M_{Ed} := \frac{Q \cdot l_{bridge}^{2}}{8} = 1.304 \times 10^{4} \cdot kN \cdot m$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.037 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.316 \times 10^{4} \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.991$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon\left(t_w\right) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119 \qquad \text{Angle of web}$$

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4 \qquad \text{(two stiffeners)}$$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15εt of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.491$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{tst} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon\left(t_s\right) \cdot \sqrt{k_{\tau 1}}} = 0.455$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$$\alpha := \frac{a}{h_w} = 1.493$$

SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9 \cdot \left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 4.782 \times 10^{3}$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.635 \times 10^{3}$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \le 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_\tau} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.455$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.898 \times 10^3 \cdot kN$

$$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.682$$

$$V_{b.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 385.532 \cdot mm$$

$$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 491.468 \cdot mm$$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot |a_{öf.eff.neg}| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot |a_{uf.eff.neg}| \cdot f_y(t_b)}{\gamma_{M0}} = 7.191 \times 10^3 \cdot kN \cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.623\,m$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.59\,m$$

$$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 294.676 \cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot kN$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.682$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.643 \times 10^{-3} \cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 432.579 \cdot cm^2$

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2.2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 9.169 \times 10^{-4}\,m^4$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Inre tvärsnittsdelar, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Höjd dragen del

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Höjd tryckt del

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_{a.v} := t_d = 10 \cdot mm$

$z_{a.v} := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2b}\left(\psi_{so}\right), h_{so}, t_{so}\right) = 0.484$

$\rho_{d.so} := \rho_{4.3}\left(\lambda_{p.so}, \psi_s\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m$ $\qquad \psi_d := \psi_{neg}\left(z_d, z_d\right) = 1$

$\delta s_{so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}\left(\psi_d\right) = 4$

$\lambda_{p.d} := \lambda_p\left(k_{\sigma.d}, s_{c.so}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.31 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.053 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^{5} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.629$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 528.504$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot \left(1 - \nu^2\right) \cdot b_d^{2}} = 818.858 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_p := \sqrt{\frac{\beta_{A.c} \cdot f_y\left(t_d\right)}{\sigma_{cr.p}}} = 0.642$

$\rho := \rho_{4.2}\left(\lambda_p, \psi\right) = 1$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, \psi\right) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 5.375 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 1.908 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{7} \cdot mm^4$

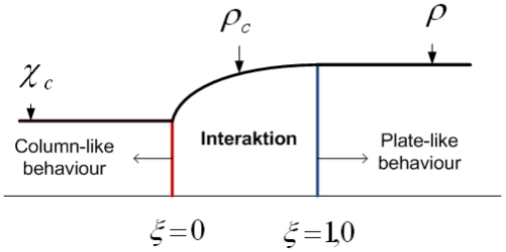
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 848.304 \cdot MPa$

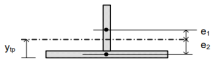
Slenderness ratio

$$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.631$$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$$

$$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$$

$$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$$

$$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^{\,2}\right] = 0.811$$ EC1993-1-1 6.3.1.2 (1)

$$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^{\,2}}}\right) = 0.757$$

Interaction

$$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.757$$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$$F_{avst} := n_{so}\, s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571 \times 10^4 \cdot kN$$

$$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so}\, s_{so.eff} \cdot t_d + A_{so}} = -319.999 \cdot MPa$$

Check according to A.2.1 (4)

$$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 268.667 \cdot MPa$$

$$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$$A_{d.eff} := n_{s.so}\, t_d \cdot s_{so.eff} = 0.032\,m^2$$

$$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 371.724 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^{\,3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^{\,3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^{\,3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.788 \times 10^{-4}\,m^4$$

$$I_{y.d.eff} := \frac{n_{s.so}\, s_{so.eff} \cdot t_d^{\,3}}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323 \times 10^{-3}\,m^4$$

$$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$$A_{öf.eff.neg} := A_{d.eff}$$

$$A_{c.eff.loc} = 5.053 \times 10^4 \cdot mm^2$$

$$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.819 \times 10^3 \cdot mm^2$$

$$A_{uf.eff.neg} := A_b$$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 414.573 \cdot cm^2$$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.076 \times 10^3 \cdot cm^2$$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d & \end{cases} = 7.568 \cdot mm$$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 396.351 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.924 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206 \cdot mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.051 \times 10^{-3}\,m^4$$

$$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4 \quad \text{from FEM}$$

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 7.557 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4 \quad \text{from FEM}$$

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.543 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.774 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 4.614 \times 10^{-4}\,m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.451 \times 10^{10} \cdot mm^4$$

Load

$q_{M} := 91\,\frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$$Q := q_{M} \cdot b_d = 318.5 \cdot \frac{kN}{m} \qquad V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.866 \times 10^3 \cdot kN$$

$$M_{Ed} := \frac{Q \cdot l_{bridge}^2}{8} = 1.29 \times 10^4 \cdot kN \cdot m$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.037 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.299 \times 10^4 \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.993$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119 \qquad \text{Angle of web}$$

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4 \qquad \text{(two stiffeners)}$$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15εt of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.465$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{tst} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.456$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$$\alpha := \frac{a}{h_w} = 1.642$$

SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 3.952 \times 10^3$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \cdot \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.362 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \le 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_\tau} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.456$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.866 \times 10^3 \cdot kN$

$$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.674$$

$$V_{b.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 391.351 \cdot mm$$

$$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 485.649 \cdot mm$$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot |a_{öf.eff.neg}| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot |a_{uf.eff.neg}| \cdot f_y(t_b)}{\gamma_{M0}} = 7.225 \times 10^3 \cdot kN \cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.685\,m$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.649\,m$$

$$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 267.887 \cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot kN$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.674$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.698 \times 10^{-3} \cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 422 \cdot cm^2$

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2.4m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 9.169 \times 10^{-4}\,m^4$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Inre tvärsnittsdelar, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Höjd dragen del

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Höjd tryckt del

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$
$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_{1v} := t_d = 10 \cdot mm$

$z_{2v} := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.484$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^{2}$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{ver} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.31 \times 10^{4} \cdot mm^{2}$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot (h_{so.eff} + t_d) = 5.053 \times 10^{4} \cdot mm^{2}$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot (1 - \nu^2)} = 3.205 \times 10^{5} \cdot mm^{4}$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \ldots + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{8} \cdot mm^{4}$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \qquad \alpha := \frac{a}{b_d} = 0.686$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02\, m^{2}$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035\, m^{2}$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[3]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 444.38$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot (1 - \nu^2) \cdot b_d^{2}} = 688.518 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_p := \sqrt{\frac{\beta_{A.c} \cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.7$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 0.979$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 5.27 \times 10^{4} \cdot mm^{2}$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 1.871 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \ldots + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{7} \cdot mm^{4}$

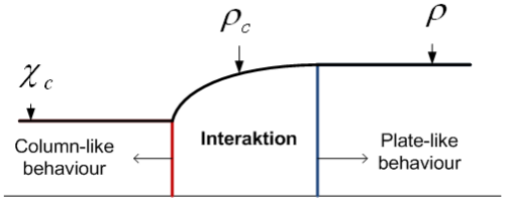
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{3} \cdot mm^{2}$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{3} \cdot mm^{2}$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

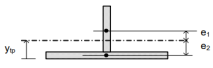
$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 712.811 \cdot MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.688$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections



$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5\cdot\left[1 + \alpha_c\cdot\left(\lambda_c - 0.2\right) + \lambda_c^{\,2}\right] = 0.864$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^{\,2}}}\right) = 0.721$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := \left(\rho - \chi_c\right)\cdot\xi\cdot(2 - \xi) + \chi_c = 0.721$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so}\cdot s_{so.eff}\cdot t_d\cdot\sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so}\cdot\sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571\times 10^{4}\cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so}\cdot s_{so.eff}\cdot t_d + A_{so}} = -319.999\cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c\cdot f_y\left(t_w\right)}{\gamma_M} = 256.05\cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c\cdot f_y\left(t_b\right)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so}\,t_d\cdot s_{so.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff}\cdot\frac{t_d}{2} + A_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2\cdot\left[\begin{aligned} &t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2}\right)\end{aligned}\right] \dots \\ &+ A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su}\cdot\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)\end{aligned}}{A_{tot.eff}} = 371.724\cdot mm$$

$$I_{y.s.eff} := 2\cdot\left[\begin{aligned} &\frac{t_s\cdot h_{s.eff.c1}^{\,3}}{12} + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s\cdot h_{s.eff.c2}^{\,3}}{12} + t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s\cdot h_{s.t}^{\,3}}{12} + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\end{aligned}\right] = 3.788\times 10^{-4}\,m^4$$

$I_{y.d.eff} := \frac{n_{s.so}\,s_{so.eff}\cdot t_d^{\,3}}{12} + n_{s.so}\cdot t_d\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323\times 10^{-3}\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053\times 10^{4}\cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c\cdot f_y\left(t_b\right)\cdot A_{sl}}{\sigma_{com.Ed}\cdot\gamma_M} = -4.593\times 10^{3}\cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{web.eff} := \begin{cases} \rho_c\cdot A_{c.eff.loc.mod} + s_{so.eff}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot A_{c.eff.loc} + s_{so.eff}\cdot t_d & \text{otherwise} \end{cases} = 396.614\cdot cm^2$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.058\times 10^{3}\cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c\cdot\frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot t_d & \end{cases} = 7.213\cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so}\cdot h_{so.eff}\cdot t_{so}\cdot\frac{t_{d.eff}}{t_d}\cdot\left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} \dots \\ &+ n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_d}{2}\right) + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su}\cdot\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)\end{aligned}}{A_{eff.tot}} = 402.489\cdot mm$$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so}\,s_{so.eff}\cdot t_d^{\,3}}{12} + t_d\cdot n_{s.so}\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.079\times 10^{-3}\,m^4$

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so}\cdot\frac{t_{so}\cdot h_{so.eff}^{\,3}}{12} + n_{so}\cdot t_{so}\cdot h_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.13\times 10^{-3}\,m^4$

$I_{y.st.su.1.fin} := 6\cdot 1.506\cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots = 7.28\times 10^{-4}\,m^4$$
$$+ 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$$
$$+ 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$$
$$+ 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp.eff.neg} \right]^2 = 2.447 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2} \right)^2 = 3.679 \times 10^{-3} m^4$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg} \right)^2 \ldots \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg} \right)^2 \ldots \\ + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg} \right)^2 \end{array} \right] = 4.837 \times 10^{-4} m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.455 \times 10^{10} \cdot mm^4$

Load

$q_M := 90 \frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$Q := q_M \cdot b_d = 315 \cdot \frac{kN}{m}$ $V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.835 \times 10^3 \cdot kN$

$M_{Ed} := \frac{Q \cdot l_{bridge}^2}{8} = 1.276 \times 10^4 \cdot kN \cdot m$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.036 \cdot m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.283 \times 10^4 \cdot kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.994$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293 m$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w} \right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180} \right) \cdot h_{su}}{2} = 0.503 m$

$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 0.913 m$

$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 1.324 m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2t_{su} \cdot h_{su} \left[(a_{st.z.1})^2 + (a_{st.z.2})^2 + (a_{st.z.3})^2 \right] = 8.18 \times 10^{-3} m^4$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) + \frac{t_{su}}{2} = 0.525 m$

$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 0.935 m$

$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 1.346 m$

Moment of inirtia of 15εt of the web z-direction

$I_{stw.z} := 2b_{web} \cdot t_w \left[(a_{st.z.1})^2 + (a_{st.z.2})^2 + (a_{st.z.3})^2 \right] = 0.021 \cdot m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029 m^4$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \quad = 5.445 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases}$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.457$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.791$

$k_{\tau sl} := \max\left[9 \left(\frac{h_w}{a} \right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{(\min(t_s, t_w))^3 \cdot h_w} \right]^3}, \frac{2.1}{\min(t_s, t_w)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}} \right] = 3.321 \times 10^3$ SS-EN 1993-1-5 (A.5)

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4 \left(\frac{h_{web}}{a} \right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min(t_s, t_w)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min(t_s, t_w)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 1.155 \times 10^3$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_1 := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$\text{Check} := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \leq 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w) \sqrt{k_\tau}}, \lambda_{w1} \right) = 0.457$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \quad = 1.2 \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases}$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.835 \times 10^{3} \cdot kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.667$

$V_{b.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 397.489 \cdot mm$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 479.511 \cdot mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot \left| a_{öf.eff.neg} \right| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot \left| a_{uf.eff.neg} \right| \cdot f_y(t_b)}{\gamma_{M0}} = 7.26 \times 10^{3} \cdot kN \cdot m$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^{2} \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^{2} \cdot f_y(t_s)} \right] = 0.747\,m$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^{2} \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^{2} \cdot f_y(t_s)} \right] = 0.708\,m$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^{2} \cdot f_y(t_d)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^{2} \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 245.563 \cdot kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}} \right)^{2} \right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^{3} \cdot kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.667$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.758 \times 10^{-3} \cdot m^{4}$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 411.206 \cdot cm^{2}$

$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2} \right) + n_{so} \cdot t_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2} \right) + A_b \cdot \left(h_{ts} - \frac{t_b}{2} \right)}{A_{eff.f.tot}} = 345.498 \cdot mm$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.025 \cdot m^{3}$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 8.999 \times 10^{3} \cdot kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.283 \times 10^{4} \cdot kN \cdot m$

Moment capacity check when $q_M = 90 \cdot \frac{kN}{m^{2}}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.994$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2.6m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 9.169 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ Table 5.2 ES1993-1-1

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Height tension part

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Height compression part

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_{sv} := t_d = 10 \cdot mm$

$z_{sv} := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.484$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^4 \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5\times10^{-3}\,m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$s_{so.eff} := s_{so} - t_{so}$ Table 5.2 ES1993-1-1 $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d\cdot s_{c.so} = 321.461\cdot mm$

Areas:

$A_{ex} := n_{so}\, s_{so}\cdot t_d + n_{so}\cdot t_{so}\cdot(h_{so.eff} + t_d) = 5.31\times10^4\cdot mm^2$

$A_{c.eff.loc} := n_{so}\cdot s_{so.eff}\cdot t_d + n_{so}\cdot t_{so}\cdot(h_{so.eff} + t_d) = 5.053\times10^4\cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^5\cdot mm^4$

$y_{tp.sl} := \frac{n_{so}\, t_{so}\cdot h_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right) + b_d\cdot t_d\cdot\frac{t_d}{2}}{n_{so}\, t_{so}\cdot h_{so} + b_d\cdot t_d} = 32.411\cdot mm$

$I_{sl} := \left[\frac{b_d\cdot t_d^3}{12} + b_d\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \ldots + \left[\frac{n_{so}\cdot t_{so}\cdot h_{so}^3}{12} + n_{so}\cdot h_{so}\cdot t_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052\times10^8\cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \qquad \alpha := \frac{a}{b_d} = 0.743$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so}\cdot h_{so}\cdot t_{so} = 0.02\,m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d\cdot t_d = 0.035\,m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2\cdot(\psi+1)\cdot(1+\delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4\cdot\left(1+\sqrt[2]{\gamma}\right)}{(\psi+1)\cdot(1+\delta)} & \text{otherwise} \end{cases} = 378.927$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p}\cdot\frac{E\cdot\pi^2\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 587.106\cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_p := \sqrt{\frac{\beta_{A.c}\cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.759$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 0.936$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi)\cdot A_{c.eff.loc} + s_{so.eff}\cdot t_d = 5.051\times10^4\cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 1.793\times10^4\cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so}\cdot h_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right) + s_{so}\cdot t_d\cdot\frac{t_d}{2}}{t_{so}\cdot h_{so} + s_{so}\cdot t_d} = 34.268\cdot mm$

$I_{sl.c} := \left[\frac{s_{so}\cdot t_d^3}{12} + s_{so}\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \ldots + \left[\frac{t_{so}\cdot h_{so}^3}{12} + h_{so}\cdot t_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137\times10^7\cdot mm^4$

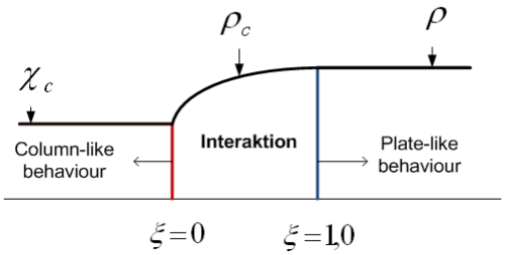
Buckling resistance stiffener

$A_{sl} := t_{so}\cdot h_{so} + s_{so}\cdot t_d = 5.74\times10^3\cdot mm^2$

$A_{sl.eff} := t_{so}\cdot h_{so.eff} + s_{so.eff}\cdot t_d = 5.455\times10^3\cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

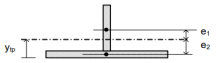
$\sigma_{cr.sl} := \frac{\pi^2\cdot E\cdot I_{sl.c}}{A_{sl}\cdot a^2} = 607.365\cdot MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c\cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.745$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_y := 0.49$ Stiffeners open cross-sections



$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589\cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5\cdot\left[1 + \alpha_c\cdot(\lambda_c - 0.2) + \lambda_c^{\,2}\right] = 0.92$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^{\,2}}}\right) = 0.685$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ is almost zero --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c)\cdot\xi\cdot(2 - \xi) + \chi_c = 0.685$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so}\, s_{so.eff}\cdot t_d\cdot\sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so}\cdot\sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571\times 10^4\cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so}\, s_{so.eff}\cdot t_d + A_{so}} = -319.999\cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c\cdot f_y(t_w)}{\gamma_M} = 243.278\cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c\cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so}\, t_d\cdot s_{so.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff}\cdot\frac{t_d}{2} + A_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2\cdot\left[\begin{aligned} &t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2}\right)\end{aligned}\right] \dots \\ &+ A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su}\cdot\left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)\end{aligned}}{A_{tot.eff}} = 371.724\cdot mm$$

$$I_{y.s.eff} := 2\cdot\left[\begin{aligned} &\frac{t_s\cdot h_{s.eff.c1}^{\,3}}{12} + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s\cdot h_{s.eff.c2}^{\,3}}{12} + t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s\cdot h_{s.t}^{\,3}}{12} + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\end{aligned}\right] = 3.788\times 10^{-4}\,m^4$$

$$I_{y.d.eff} := \frac{n_{s.so}\, s_{so.eff}\cdot t_d^{\,3}}{12} + n_{s.so}\cdot t_d\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323\times 10^{-3}\,m^4$$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053\times 10^4\cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c\cdot f_y(t_b)\cdot A_{sl}}{\sigma_{com.Ed}\cdot\gamma_M} = -4.364\times 10^3\cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{w.eff} := \begin{cases} \rho_c\cdot A_{c.eff.loc.mod} + s_{so.eff}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot A_{c.eff.loc} + s_{so.eff}\cdot t_d & \text{otherwise} \end{cases} = 378.433\cdot cm^2$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.04\times 10^3\cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c\cdot\frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot t_d & \end{cases} = 6.853\cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so}\cdot h_{so.eff}\cdot t_{so}\cdot\frac{t_{d.eff}}{t_d}\cdot\left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} \dots \\ &+ n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_d}{2}\right) + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su}\cdot\left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)\end{aligned}}{A_{eff.tot}} = 408.918\cdot mm$$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so}\, s_{so.eff}\cdot t_d^{\,3}}{12} + t_d\cdot n_{s.so}\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.245\times 10^{-3}\,m^4$

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so}\cdot\frac{t_{so}\cdot h_{so.eff}^{\,3}}{12} + n_{so}\cdot t_{so}\cdot h_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.214\times 10^{-3}\,m^4$

$I_{y.st.su.1.fin} := 6\cdot 1.506\cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots = 6.998\times 10^{-4}\,m^4$$
$$+ 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$$
$$+ 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$$
$$+ 2t_{su}\cdot h_{su}\cdot\left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp.eff.neg} \right]^2 = 2.35 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2} \right)^2 = 3.581 \times 10^{-3} m^4$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg} \right)^2 \ldots \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg} \right)^2 \ldots \\ + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg} \right)^2 \end{array} \right] = 5.078 \times 10^{-4} m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.46 \times 10^{10} \cdot mm^4$

Load

$q_M := 89 \frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$Q := q_M \cdot b_d = 311.5 \cdot \frac{kN}{m}$ $V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.804 \times 10^3 \cdot kN$

$M_{Ed} := \frac{Q \cdot l_{bridge}^2}{8} = 1.262 \times 10^4 \cdot kN \cdot m$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.036 \cdot m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.267 \times 10^4 \cdot kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.995$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293 m$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w} \right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180} \right) \cdot h_{su}}{2} = 0.503 m$

$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 0.913 m$

$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 1.324 m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 t_{su} \cdot h_{su} \cdot \left[(a_{st.z.1})^2 + (a_{st.z.2})^2 + (a_{st.z.3})^2 \right] = 8.18 \times 10^{-3} m^4$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) + \frac{t_{su}}{2} = 0.525 m$

$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 0.935 m$

$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 1.346 m$

Moment of inirtia of 15εt of the web z-direction

$I_{stw.z} := 2 b_{web} \cdot t_w \cdot \left[(a_{st.z.1})^2 + (a_{st.z.2})^2 + (a_{st.z.3})^2 \right] = 0.021 \cdot m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029 m^4$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.43$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.457$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha_w := \frac{a}{h_w} = 1.94$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9 \left(\frac{h_w}{a} \right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{(\min(t_s, t_w))^3 \cdot h_w} \right]^3}, \frac{2.1}{\min(t_s, t_w)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}} \right] = 2.829 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4 \left(\frac{h_{web}}{a} \right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min(t_s, t_w)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min(t_s, t_w)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 993.262$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \leq 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w) \sqrt{k_\tau}}, \lambda_{w1} \right) = 0.457$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.804 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.66$

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 403.918\cdot mm$$

$$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 473.082\cdot mm$$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg}\cdot\left|a_{öf.eff.neg}\right|\cdot f_y(t_d)}{\gamma_{M0}} \ldots = 7.297\times 10^3\cdot kN\cdot m$$
$$+ \frac{A_{uf.eff.neg}\cdot\left|a_{uf.eff.neg}\right|\cdot f_y(t_b)}{\gamma_{M0}}$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$$M_{Ed} < M_{f.Rd.neg} = 0$$

$$c_{öf} := a\cdot\left[0.25 + \frac{1.6\cdot b_d\cdot t_d^2\cdot f_y(t_d)}{t_s\cdot(h_s + h_{wy})^2\cdot f_y(t_s)}\right] = 0.81\,m$$

$$c_{uf} := a\cdot\left[0.25 + \frac{1.6\cdot b_b\cdot t_b^2\cdot f_y(t_b)}{t_s\cdot(h_s + h_{wy})^2\cdot f_y(t_s)}\right] = 0.767\,m$$

$$V_{bf.Rd.ored} := \frac{b_d\cdot t_d^2\cdot f_y(t_d)}{c_{öf}\cdot\gamma_{M1}\cdot\gamma_n} + \frac{b_b\cdot t_b^2\cdot f_y(t_b)}{c_{uf}\cdot\gamma_{M1}\cdot\gamma_n} = 226.674\cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored}\cdot\left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0\cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta\cdot f_y(t_s)\cdot h_{web}\cdot t_s}{\sqrt{3}\cdot\gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125\times 10^3\cdot kN$$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2V_{b.Rd}} = 0.66$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.826\times 10^{-3}\cdot m^4$$

The flanges effective area:

$$A_{eff.f.tot} := s_{so.eff}\cdot t_d + n_{so}\cdot s_{so.eff}\cdot t_{d.eff} + n_{so}\cdot t_{so}\cdot t_{d.eff} + A_b = 400.279\cdot cm^2$$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} + n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right)\ldots}{+ n_{so}\cdot t_{so}\cdot t_{d.eff}\cdot\left(\frac{t_{d.eff}}{2}\right) + A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)} = 354.738\cdot mm$$
$$A_{eff.f.tot}$$

$$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.025\cdot m^3$$

The plastic moment bearing capacity calculated with the flanges effective area:

$$M_{f.Rd} := W_{f.y.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 8.833\times 10^3\cdot kN\cdot m$$

Results

$$M_{pl.Rd} := M_{Rd} = 1.267\times 10^4\cdot kN\cdot m$$

Moment capacity check when $q_M = 89\cdot\frac{kN}{m^2}$

$$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.995$$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 2.8m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 9.169 \times 10^{-4}\,m^4$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Inre tvärsnittsdelar, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Höjd dragen del

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Höjd tryckt del

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_{a.v} := t_d = 10 \cdot mm$

$z_{a.v} := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2b}\left(\psi_{so}\right), h_{so}, t_{so}\right) = 0.484$

$\rho_{d.so} := \rho_{4.3}\left(\lambda_{p.so}, \psi_s\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m \qquad \psi_d := \psi_{neg}\left(z_d, z_d\right) = 1$

$\delta_{so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}\left(\psi_d\right) = 4$

$\lambda_{p.d} := \lambda_p\left(k_{\sigma.d}, s_{c.so}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.31 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.053 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^{5} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \qquad \alpha := \frac{a}{b_d} = 0.8$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 327.007$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot \left(1 - \nu^2\right) \cdot b_d^{2}} = 506.661 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_p := \sqrt{\frac{\beta_{A.c} \cdot f_y\left(t_d\right)}{\sigma_{cr.p}}} = 0.817$

$\rho := \rho_{4.2}\left(\lambda_p, \psi\right) = 0.895$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, \psi\right) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 4.843 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 1.719 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{7} \cdot mm^4$

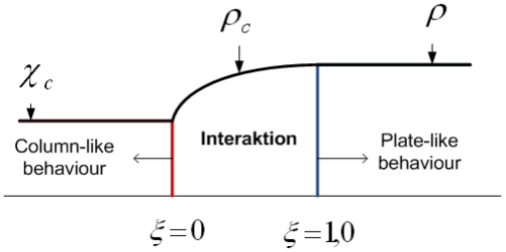
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 523.698 \cdot MPa$

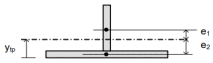
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.803$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 0.979$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.649$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.649$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -319.999 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 230.475 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 371.724 \cdot mm$

$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.788 \times 10^{-4}\,m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323 \times 10^{-3}\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -4.134 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 360.209 \cdot cm^2$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.022 \times 10^3 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d \end{cases} = 6.492 \cdot mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 415.592 \cdot mm$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.42 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.303 \times 10^{-3}\,m^4$$

$$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4 \quad \text{from FEM}$$

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 6.711 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4 \quad \text{from FEM}$$

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.252 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.481 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 5.337 \times 10^{-4}\,m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.466 \times 10^{10} \cdot mm^4$$

Load

$q_M := 88\,\frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$$Q := q_M \cdot b_d = 308 \cdot \frac{kN}{m} \qquad V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.772 \times 10^{3} \cdot kN$$

$$M_{Ed} := \frac{Q \cdot l_{bridge}^{2}}{8} = 1.247 \times 10^{4} \cdot kN \cdot m$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.035 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.252 \times 10^{4} \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.996$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119 \qquad \text{Angle of web}$$

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4 \qquad \text{(two stiffeners)}$$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$$a_{stw.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15εt of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.417$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.458$$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$$\alpha := \frac{a}{h_w} = 2.09$$

SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 2.44 \times 10^{3}$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 865.145$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \le 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_\tau} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.458$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.772 \times 10^3 \cdot kN$

$$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.652$$

$$V_{b.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 410.592 \cdot mm$$

$$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 466.408 \cdot mm$$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot |a_{öf.eff.neg}| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot |a_{uf.eff.neg}| \cdot f_y(t_b)}{\gamma_{M0}} = 7.335 \times 10^3 \cdot kN \cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.872\,m$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.826\,m$$

$$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 210.483 \cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot kN$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.652$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.901 \times 10^{-3} \cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 389.325 \cdot cm^2$

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 3m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 9.169 \times 10^{-4}\,m^4$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ Tab. 5.2 ES1993-1-1

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Internal parts, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Height, in tension

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Height, in compression

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_{sw} := t_d = 10 \cdot mm$

$z_{sw} := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p(k_{\sigma.4.2b}(\psi_{so}), h_{so}, t_{so}) = 0.484$

$\rho_{d.so} := \rho_{4.3}(\lambda_{p.so}, \psi_s) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^4 \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42\cdot\sqrt{\frac{235MPa}{f_y(t_d)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5\times10^{-3}\,m \qquad \psi_d := \psi_{neg}(z_d, z_d) = 1$

$s_{so.eff} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}(\psi_d) = 4$

$\lambda_{p.d} := \lambda_p(k_{\sigma.d}, s_{c.so}, t_d) = 0.723$

$\rho_d := \rho_{4.2}(\lambda_{p.d}, \psi_d) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d\cdot s_{c.so} = 321.461\cdot mm$

Areas:

$A_{sl} := n_{so}\, s_{so}\cdot t_d + n_{so}\cdot t_{so}\cdot(h_{so.eff} + t_d) = 5.31\times10^4\cdot mm^2$

$A_{c.eff.loc} := n_{so}\cdot s_{so.eff}\cdot t_d + n_{so}\cdot t_{so}\cdot(h_{so.eff} + t_d) = 5.053\times10^4\cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d\cdot t_d^3}{12\cdot(1-\nu^2)} = 3.205\times10^5\cdot mm^4$

$y_{tp.sl} := \frac{n_{so}\, t_{so}\cdot h_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right) + b_d\cdot t_d\cdot\frac{t_d}{2}}{n_{so}\, t_{so}\cdot h_{so} + b_d\cdot t_d} = 32.411\cdot mm$

$I_{sl} := \left[\frac{b_d\cdot t_d^3}{12} + b_d\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] \ldots + \left[\frac{n_{so}\cdot t_{so}\cdot h_{so}^3}{12} + n_{so}\cdot h_{so}\cdot t_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052\times10^8\cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \qquad \alpha := \frac{a}{b_d} = 0.857$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so}\cdot h_{so}\cdot t_{so} = 0.02\,m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d\cdot t_d = 0.035\,m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2\cdot\left[\left(1+\alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2\cdot(\psi+1)\cdot(1+\delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4\cdot\left(1+\sqrt{\gamma}\right)}{(\psi+1)\cdot(1+\delta)} & \text{otherwise} \end{cases} = 285.135$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p}\cdot\frac{E\cdot\pi^2\cdot t_d^2}{12\cdot(1-\nu^2)\cdot b_d^2} = 441.786\cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_p := \sqrt{\frac{\beta_{A.c}\cdot f_y(t_d)}{\sigma_{cr.p}}} = 0.874$

$\rho := \rho_{4.2}(\lambda_p, \psi) = 0.856$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}(\lambda_p, \psi)\cdot A_{c.eff.loc} + s_{so.eff}\cdot t_d = 4.646\times10^4\cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff}\cdot\frac{f_y(t_d)}{\gamma_M} = 1.649\times10^4\cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so}\cdot h_{so}\cdot\left(t_d + \frac{h_{so}}{2}\right) + s_{so}\cdot t_d\cdot\frac{t_d}{2}}{t_{so}\cdot h_{so} + s_{so}\cdot t_d} = 34.268\cdot mm$

$I_{sl.c} := \left[\frac{s_{so}\cdot t_d^3}{12} + s_{so}\cdot t_d\cdot\left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] \ldots + \left[\frac{t_{so}\cdot h_{so}^3}{12} + h_{so}\cdot t_{so}\cdot\left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137\times10^7\cdot mm^4$

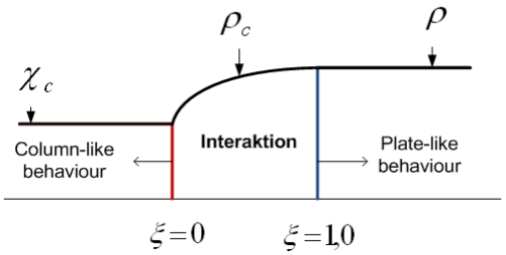
Buckling resistance stiffener

$A_{sl} := t_{so}\cdot h_{so} + s_{so}\cdot t_d = 5.74\times10^3\cdot mm^2$

$A_{sl.eff} := t_{so}\cdot h_{so.eff} + s_{so.eff}\cdot t_d = 5.455\times10^3\cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

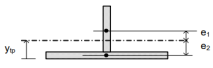
$\sigma_{cr.sl} := \frac{\pi^2\cdot E\cdot I_{sl.c}}{A_{sl}\cdot a^2} = 456.199\cdot MPa$

Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c\cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.86$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha_e := 0.49$ Stiffeners open cross-sections



$e = 2.718$

$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362\cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589\cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5\cdot\left[1 + \alpha_c\cdot\left(\lambda_c - 0.2\right) + \lambda_c^{2}\right] = 1.042$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^{2}}}\right) = 0.613$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := \left(\rho - \chi_c\right)\cdot\xi\cdot(2 - \xi) + \chi_c = 0.613$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so}\, s_{so.eff}\cdot t_d\cdot\sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so}\cdot\sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571\times 10^{4}\cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so}\, s_{so.eff}\cdot t_d + A_{so}} = -319.999\cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c\cdot f_y\left(t_w\right)}{\gamma_M} = 217.781\cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c\cdot f_y\left(t_b\right)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so}\, t_d\cdot s_{so.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff}\cdot\frac{t_d}{2} + A_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2\cdot\left[\begin{aligned} &t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2}\right)\end{aligned}\right] \dots \\ &+ A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su}\cdot\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)\end{aligned}}{A_{tot.eff}} = 371.724\cdot mm$$

$$I_{y.s.eff} := 2\cdot\left[\begin{aligned} &\frac{t_s\cdot h_{s.eff.c1}^{3}}{12} + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s\cdot h_{s.eff.c2}^{3}}{12} + t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s\cdot h_{s.t}^{3}}{12} + t_s\cdot h_{s.t}\left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2\end{aligned}\right] = 3.788\times 10^{-4}\,m^4$$

$I_{y.d.eff} := \frac{n_{s.so}\, s_{so.eff}\cdot t_d^{3}}{12} + n_{s.so}\cdot t_d\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323\times 10^{-3}\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053\times 10^{4}\cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c\cdot f_y\left(t_b\right)\cdot A_{sl}}{\sigma_{com.Ed}\cdot\gamma_M} = -3.906\times 10^{3}\cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c\cdot A_{c.eff.loc.mod} + s_{so.eff}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot A_{c.eff.loc} + s_{so.eff}\cdot t_d & \text{otherwise} \end{cases} = 342.14\cdot cm^2$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 1.004\times 10^{3}\cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$$t_{d.eff} := \begin{cases} \rho_c\cdot\frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}}\cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c\cdot t_d & \end{cases} = 6.135\cdot mm$$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so}\cdot h_{so.eff}\cdot t_{so}\cdot\frac{t_{d.eff}}{t_d}\cdot\left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff}\cdot t_d\cdot\frac{t_d}{2} \dots \\ &+ n_{so}\cdot s_{so.eff}\cdot t_{d.eff}\cdot\left(\frac{t_d}{2}\right) + t_s\cdot h_{s.eff.c1}\cdot\left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s\cdot h_{s.eff.c2}\cdot\left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s\cdot h_{s.t}\cdot\left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w\cdot\left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su}\cdot\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0}\right) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b\cdot\left(h_{ts} - \frac{t_b}{2}\right)\end{aligned}}{A_{eff.tot}} = 422.448\cdot mm$$

Effective moment of inertia:

$I_{y.d.eff.fin} := \frac{n_{s.so}\, s_{so.eff}\cdot t_d^{3}}{12} + t_d\cdot n_{s.so}\cdot s_{so.eff}\cdot\left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.602\times 10^{-3}\,m^4$

$s_{su0y} := \sin\left(21.12\cdot\frac{\pi}{180}\right)\cdot s_{su0} = 7.206\cdot mm$

Stiffeners over/under:

$I_{y.st.so.fin} := n_{so}\cdot\frac{t_{so}\cdot h_{so.eff}^{3}}{12} + n_{so}\cdot t_{so}\cdot h_{so.eff}\cdot\left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.397\times 10^{-3}\,m^4$

$I_{y.st.su.1.fin} := 6\cdot 1.506\cdot 10^{-6}\,m^4$ from FEM

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \dots = 6.426\times 10^{-4}\,m^4$$
$$+ 2t_{su}\cdot h_{su}\left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$$
$$+ 2t_{su}\cdot h_{su}\left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \dots$$
$$+ 2t_{su}\cdot h_{su}\left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$I_{y.w.1.fin} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w.fin} := I_{y.w.1.fin} + 2 t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp.eff.neg} \right]^2 = 2.154 \times 10^{-3} m^4$

Bottom flange:

$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2} \right)^2 = 3.379 \times 10^{-3} m^4$

Side plate:

$$I_{y.s.fin} := 2 \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg} \right)^2 \ldots \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg} \right)^2 \ldots \\ + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg} \right)^2 \end{array} \right] = 5.612 \times 10^{-4} m^4$$

$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.474 \times 10^{10} \cdot mm^4$

Load

$q_M := 87 \frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$Q := q_M \cdot b_d = 304.5 \cdot \frac{kN}{m}$ $V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.74 \times 10^3 \cdot kN$

$M_{Ed} := \frac{Q \cdot l_{bridge}^2}{8} = 1.233 \times 10^4 \cdot kN \cdot m$

Moment capacity

$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.035 \cdot m^3$

$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.238 \times 10^4 \cdot kN \cdot m$

$\frac{M_{Ed}}{M_{Rd}} = 0.996$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293 m$

$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w} \right) \cdot \frac{180}{\pi} = 21.119$ Angle of web

Distance to stiffeners from z-axis:

$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180} \right) \cdot h_{su}}{2} = 0.503 m$

$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 0.913 m$

$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 1.324 m$

Moment of inirtia stiffeners z-direction

$I_{st.z} := 2 t_{su} \cdot h_{su} \cdot \left[(a_{st.z.1})^2 + (a_{st.z.2})^2 + (a_{st.z.3})^2 \right] = 8.18 \times 10^{-3} m^4$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$a_{stw.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) + \frac{t_{su}}{2} = 0.525 m$

$a_{stw.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 0.935 m$

$a_{stw.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180} \right) = 1.346 m$

Moment of inirtia of 15εt of the web z-direction

$I_{stw.z} := 2 b_{web} \cdot t_w \cdot \left[(a_{st.z.1})^2 + (a_{st.z.2})^2 + (a_{st.z.3})^2 \right] = 0.021 \cdot m^4$

$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029 m^4$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a} \right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.407$$ SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.458$ EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$\alpha := \frac{a}{h_w} = 2.239$ SS-EN 1993-1-5 (A.5)

$k_{\tau sl} := \max\left[9 \left(\frac{h_w}{a} \right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min(t_s, t_w) \right)^3 \cdot h_w} \right]^3}, \frac{2.1}{\min(t_s, t_w)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}} \right] = 2.125 \times 10^3$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4 \left(\frac{h_{web}}{a} \right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min(t_s, t_w)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min(t_s, t_w)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 761.787$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_1 := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \leq 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_{\tau 1}} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w) \sqrt{k_\tau}}, \lambda_{w1} \right) = 0.458$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \leq \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \geq 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_w := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.74 \times 10^3 \cdot kN$

$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.645$

$V_{b.Rd} \leq \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 417.448 \cdot mm$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 459.552 \cdot mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot |a_{öf.eff.neg}| \cdot f_y(t_d)}{\gamma_{M0}} \ldots = 7.374 \times 10^3 \cdot kN \cdot m$
$+ \frac{A_{uf.eff.neg} \cdot |a_{uf.eff.neg}| \cdot f_y(t_b)}{\gamma_{M0}}$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)} \right] = 0.934\,m$

$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)} \right] = 0.885\,m$

$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 196.451 \cdot kN$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}} \right)^2 \right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$

Shear capacity stiffened web:

$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot kN$

Check:

$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$ EC1993-1-5, Ch.5.5, eq. 5.10

$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.645$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 8.981 \times 10^{-3} \cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 378.465 \cdot cm^2$

$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2} \right) \ldots}{+ n_{so} \cdot t_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2} \right) + A_b \cdot \left(h_{ts} - \frac{t_b}{2} \right)} = 374.809 \cdot mm$
$A_{eff.f.tot}$

$W_{f.y.eff} := \frac{I_{y.f.eff.fin}}{z_{tp.f.eff.neg}} = 0.024 \cdot m^3$

The plastic moment bearing capacity calculated with the flanges effective area:

$M_{f.Rd} := W_{f.y.eff} \cdot \frac{f_y(t_d)}{\gamma_M} = 8.507 \times 10^3 \cdot kN \cdot m$

Results

$M_{pl.Rd} := M_{Rd} = 1.238 \times 10^4 \cdot kN \cdot m$

Moment capacity check when $q_M = 87 \cdot \frac{kN}{m^2}$

$\eta_1 := \frac{M_{Ed}}{M_{pl.Rd}} = 0.996$ EC1993-1-5 eq. 4.15, EC1993-1-5 Ch.7.1(5)

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 3.2m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \ldots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4$ from FEM

$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots = 9.169 \times 10^{-4}\,m^4$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \ldots$

$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$

Web:

$I_{y.w.1} := 2 \cdot 0.000312m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \quad = \text{"Lower than class 4"} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Inre tvärsnittsdelar, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Höjd dragen del

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Höjd tryckt del

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_w := t_d = 10 \cdot mm$

$z_a := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \quad = \text{"Lower than class 4"} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases}$$

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2b}\left(\psi_{so}\right), h_{so}, t_{so}\right) = 0.484$

$\rho_{d.so} := \rho_{4.3}\left(\lambda_{p.so}, \psi_s\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^4 \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m \qquad \psi_d := \psi_{neg}\left(z_d, z_d\right) = 1$

$\delta s_{so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $\qquad k_{\sigma.d} := k_{\sigma.4.1}\left(\psi_d\right) = 4$

$\lambda_{p.d} := \lambda_p\left(k_{\sigma.d}, s_{c.so}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.31 \times 10^4 \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.053 \times 10^4 \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^3}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^5 \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^3}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^8 \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.914$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt[2]{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 250.882$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^2}{12 \cdot \left(1 - \nu^2\right) \cdot b_d^2} = 388.713 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{p.p} := \sqrt{\frac{\beta_{A.c} \cdot f_y\left(t_d\right)}{\sigma_{cr.p}}} = 0.932$

$\rho := \rho_{4.2}\left(\lambda_p, \psi\right) = 0.82$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, \psi\right) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 4.463 \times 10^4 \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 1.584 \times 10^4 \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^3}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^3}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^7 \cdot mm^4$

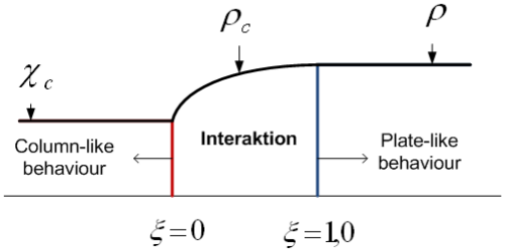
Buckling resistance stiffener

$A_{sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^3 \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^3 \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 400.956 \cdot MPa$

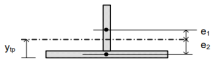
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.917$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 1.108$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.578$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.578$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -319.999 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 205.338 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$

$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 371.724 \cdot mm$

$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.788 \times 10^{-4}\,m^4$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323 \times 10^{-3}\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -3.683 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 324.429 \cdot cm^2$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 985.837 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d \end{cases} = 5.784 \cdot mm$

$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 429.413 \cdot mm$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.791 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) \cdot s_{su0} = 7.206 \cdot mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.494 \times 10^{-3}\,m^4$$

$$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4 \quad \text{from FEM}$$

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 6.144 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4 \quad \text{from FEM}$$

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 2.058 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.278 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 5.9 \times 10^{-4}\,m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.483 \times 10^{10} \cdot mm^4$$

Load

$q_M := 86\,\frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$$Q := q_M \cdot b_d = 301 \cdot \frac{kN}{m} \qquad V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.709 \times 10^3 \cdot kN$$

$$M_{Ed} := \frac{Q \cdot l_{bridge}^2}{8} = 1.219 \times 10^4 \cdot kN \cdot m$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.035 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.226 \times 10^4 \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.995$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119 \qquad \text{Angle of web}$$

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$$a_{stw.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stw.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stw.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15εt of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.399$$

SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.459$$

EC1-3-5 eq.5.6

Shear buckling coefficient; web with more than two longitudinal stiffeners

$$\alpha_{sl} := \frac{a}{h_w} = 2.388$$

SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9\left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min(t_s, t_w)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min(t_s, t_w)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 1.868 \times 10^3$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min(t_s, t_w)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min(t_s, t_w)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 677.196$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta_w := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \le 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_\tau 1} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.459$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.709 \times 10^3 \cdot kN$

$$\frac{V_{Ed}}{2 V_{bw.Rd}} = 0.637$$

$$V_{b.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 424.413 \cdot mm$$

$$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 452.587 \cdot mm$$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot |a_{öf.eff.neg}| \cdot f_y(t_d)}{\gamma_{M0}} \dots + \frac{A_{uf.eff.neg} \cdot |a_{uf.eff.neg}| \cdot f_y(t_b)}{\gamma_{M0}} = 7.414 \times 10^3 \cdot kN \cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.996\,m$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 0.944\,m$$

$$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 184.173 \cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{b.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot kN$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2 V_{b.Rd}} = 0.637$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 9.068 \times 10^{-3} \cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 367.82 \cdot cm^2$

$$z_{tp.f.eff.neg} := \frac{s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) \dots + n_{so} \cdot t_{so} \cdot t_{d.eff} \cdot \left(\frac{t_{d.eff}}{2}\right) + A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right)}{A_{eff.f.tot}} = 385.484 \cdot mm$$

EWM - vertical stiffener distance

This chapter covers the EWM when changing the vertical stiffener distance.

Geometry

Amount of:

upper stiffeners: $n_{so} := 9$

Lower stiffeners: $n_{su} := 6$

Deck subpanels: $n_{s.so} := n_{so} + 1 = 10$

Deck: $b_d := 3500mm$ $t_d := 10mm$

$b_{d.0.5} := 1750mm$

$s_{so} := \frac{b_d}{n_{s.so}} = 350 \cdot mm$

side plate: $h_s := 389.2mm$ $t_s := 12mm$

Web: $h_w := 1340mm$ $t_w := 12mm$

$h_{wy} := 482.8mm$

$h_{wx} := 1250mm$

$s_{su0} := 20mm$

$s_{su} := 440mm$

Bottom plate: $b_b := 1000mm$ $t_b := 16mm$

Longitudinal stiffeners:

Stiffeners over: $h_{so} := 140mm$ $t_{so} := 16mm$ $a_{so} := 6mm$

Stiffeners under: $h_{su} := 120mm$ $t_{su} := 12mm$ $a_{su} := 5mm$

$h_{su.y} := h_{su} \cdot \cos\left(21.12 \cdot \frac{\pi}{180}\right) = 111.939 \cdot mm$

Total height: $h_{ts} := t_d + h_s + h_{wy} + \frac{t_b}{2} = 890 \cdot mm$

Bridge length: $l_{bridge} := 18m$

Distance between vertical stiffeners: $a := 3.4m$

Transverse stiffeners: $s_{ts} := 2000mm$ $s_{ts0} := 500mm$

$t_{ts.1} := 18mm$ $t_{ts2} := 12mm$ $t_{ts3} := 8mm$

$h_{web} := h_{wy} + h_s - \frac{t_b}{2} = 864 \cdot mm$

Poissons ratio $\nu := 0.3$

$\gamma_{M1} := 1.00$ $\gamma_n := 1.2$

$\gamma_{M0} := 1.00$

$\gamma_M := 1$

Material parameters:

$E := 210GPa$

$f_y(t) := \begin{cases} (355MPa) & \text{if } t \le 40mm \\ (335MPa) & \text{otherwise} \end{cases}$ EC1993-1-1 tab. 3.1

$\varepsilon(t) := \sqrt{\frac{235MPa}{f_y(t)}}$

Area

$A_d := t_d \cdot b_d$ $A_s := 2t_s \cdot h_s$ $A_w := 2t_w \cdot h_w$ $A_b := t_b \cdot b_b$

$A_{so} := n_{so} \cdot t_{so} \cdot h_{so}$ $A_{su} := n_{su} \cdot t_{su} \cdot h_{su}$

$A_{tot} := A_d + A_s + A_w + A_b + A_{so} + A_{su} = 0.121 \cdot m^2$

$A_c := A_{so} + A_d = 0.055\,m^2$

Change of center of gravity, z_{tp}

$$z_{tp} := \frac{A_d \cdot \frac{t_d}{2} + A_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + A_s \cdot \left(t_d + \frac{h_s}{2}\right) + A_w \left(t_d + h_s + \frac{h_{wy}}{2}\right) \dots + A_b \cdot (t_d + h_s + h_{wy}) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right]}{A_{tot}} = 363.096 \cdot mm$$

From top to centre of gravity

Moment of inertia I_y

$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$

Stiffeners over/under:

$I_{y.st.so} := n_{so} \cdot \frac{t_{so} \cdot h_{so}^3}{12} + n_{so} \cdot t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2} - z_{tp}\right)^2 = 1.649 \times 10^{-3}\,m^4$

$I_{y.st.su.1} := 6 \cdot 1.506 \cdot 10^{-6} m^4$ from FEM

$$I_{y.st.su} := I_{y.st.su.1} + 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots = 9.169 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2 \dots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + (h_{wy} - s_{su0y}) - \frac{h_{su.y}}{2} - z_{tp}\right]^2$$

Web:

$I_{y.w.1} := 2 \cdot 0.000312 m^4$ from FEM

$I_{y.w} := I_{y.w.1} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot (h_{wy}) - z_{tp}\right]^2 = 3.101 \times 10^{-3}\,m^4$

Deck:

$I_{y.d} := \frac{b_d \cdot t_d^3}{12} + t_d \cdot b_d \cdot \left(\frac{t_d}{2} - z_{tp}\right)^2 = 4.488 \times 10^{-3}\,m^4$

Bottom flange:

$I_{y.b} := \frac{b_b \cdot t_b^3}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp} - \frac{t_b}{2}\right)^2 = 4.309 \times 10^{-3}\,m^4$

Side plate:

$I_{y.s} := 2\frac{t_s \cdot h_s^3}{12} + 2t_s \cdot h_s \cdot \left(t_d + \frac{h_s}{2} - z_{tp}\right)^2 = 3.526 \times 10^{-4}\,m^4$

Total:

$I_{y.tot} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.d} + I_{y.b} + I_{y.s} = 0.015\,m^4$

Reduced cross section

Functions to investigate the stresses in relevant cross sections

Stresses

$\sigma_{max} := f_y(t_d) = 355 \cdot MPa$

Stress at z-coordinate when lower flange acts in tension:

$\sigma_{neg}(z) := \frac{\sigma_{max}}{z_{tp}} \cdot z - \sigma_{max}$

$\sigma_{1.neg}(z_1, z_2) := \max(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

$\sigma_{2.neg}(z_1, z_2) := \min(\sigma_{neg}(z_1), \sigma_{neg}(z_2))$

Stress at z-coordinate when upper flange acts in tension:

$\sigma_{pos}(z) := \frac{-\sigma_{max}}{z_{tp}} \cdot z + \sigma_{max}$

$\sigma_{1.pos}(z_1, z_2) := \max(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

$\sigma_{2.pos}(z_1, z_2) := \min(\sigma_{pos}(z_1), \sigma_{pos}(z_2))$

Buckling factor

$\psi_{neg}(z_1, z_2) := \frac{\sigma_{2.neg}(z_1, z_2)}{\sigma_{1.neg}(z_1, z_2)}$ $\psi_{pos}(z_1, z_2) := \frac{\sigma_{2.pos}(z_1, z_2)}{\sigma_{1.pos}(z_1, z_2)}$

$$k_{\sigma.4.1}(\psi) := \begin{cases} 4 & \text{if } \psi = 1 \\ \frac{8.2}{1.05 + \psi} & \text{if } 1 > \psi > 0 \\ 7.81 & \text{if } \psi = 0 \\ 7.81 - 6.29\psi + 9.78\psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.98(1 - \psi)^2 & \text{if } -1 > \psi > -3 \end{cases}$$

Buckling factors from table 4.1
SS-EN 1993-1-5
Internal parts in conpression

$$k_{\sigma.4.2a}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ 0.57 & \text{if } \psi = 0 \\ 0.85 & \text{if } \psi = -1 \\ 0.57 - 0.21\psi + 0.07\psi^2 & \text{if } -1 > \psi \geq -3 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 upper part
Elements in compression with free edge.
Lower stress close to support.

$$k_{\sigma.4.2b}(\psi) := \begin{cases} 0.43 & \text{if } \psi = 1 \\ \frac{0.578}{\psi + 0.34} & \text{if } 1 > \psi > 0 \\ 1.7 & \text{if } \psi = 0 \\ 1.7 - 5\psi + 17.1\psi^2 & \text{if } 0 > \psi > -1 \\ 23.8 & \text{if } \psi = -1 \end{cases}$$

Buckling factors from table 4.2
SS-EN 1993-1-5 lower part
Elements in compression free edge.
Higher stress close to support.

Slenderness

$\lambda_p(k_\sigma, b, t) := \frac{\frac{b}{t}}{28.4 \cdot \varepsilon(t) \cdot \sqrt{k_\sigma}}$

Reduduction factors

$$\rho_{4.2}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \\ \min\left[\frac{\lambda_p - 0.055 \max[(3 + \psi), 0]}{\lambda_p^2}, 1\right] & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.2)
Internal parts

$$\rho_{4.3}(\lambda_p, \psi) := \begin{cases} 1.0 & \text{if } \lambda_p \leq 0.748 \\ \min\left(\frac{\lambda_p - 0.188}{\lambda_p^2}, 1\right) & \text{otherwise} \end{cases}$$

SS-EN 1993-1-5 (4.3)
Cross-sections with free edge

Reduced cross sections

Side plate

$z_1 := t_d = 10 \cdot mm$

$z_2 := t_d + h_s = 399.2 \cdot mm$

$\psi_s := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = -0.102$

$$Condition := \begin{cases} 62 \cdot \varepsilon(t_s) \cdot (1 - \psi_s) \cdot \sqrt{-\psi_s} & \text{if } \psi_s \leq -1 \\ \frac{42 \cdot \varepsilon(t_s)}{0.67 + 0.33 \cdot \psi_s} & \text{otherwise} \end{cases}$$

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_s}{t_s} > Condition \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$k_{\sigma.4.1}(\psi_s) = 8.555$

$s_{c.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1]

$\lambda_{p.s} := \lambda_p(k_{\sigma.4.1}(\psi_s), h_s, t_s) = 0.48$

$\rho_s := \rho_{4.2}(\lambda_{p.s}, \psi_s) = 1$ Inre tvärsnittsdelar, EC1993-1-5 4.4 (2)

$h_{s.t} := t_d + h_s - z_{tp} = 0.036\,m$ Höjd dragen del

$h_{s.c} := h_s - h_{s.t} = 353.096 \cdot mm$ Höjd tryckt del

$h_{s.eff.c} := \rho_s \cdot (h_{s.c}) = 353.096 \cdot mm$

$h_{s.eff.c1} := 0.4 \cdot h_{s.eff.c} = 141.238 \cdot mm$

$h_{s.eff.c2} := 0.6 \cdot h_{s.eff.c} = 211.858 \cdot mm$

$h_{s.eff} := h_{s.eff.c} + h_{s.t} = 389.2 \cdot mm$

$A_{s.eff.c} := 2 \cdot h_{s.eff.c} \cdot t_s = 8.474 \times 10^3 \cdot mm^2$

$A_{s.eff} := 2(h_{s.eff.c} + h_{s.t}) \cdot t_s = 9.341 \times 10^3 \cdot mm^2$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.p} := k_{\sigma.4.1}(\psi_s) \cdot \frac{E \cdot \pi^2 \cdot t_s^2}{12 \cdot (1 - \nu^2) \cdot h_s^2} = 1.544 \times 10^3 \cdot MPa$ EC1993-1-5 eq A.1

Stiffeners

$z_w := t_d = 10 \cdot mm$

$z_2 := t_d + h_{so} = 150 \cdot mm$

$\psi_{so} := \frac{\sigma_{neg}(z_2)}{\sigma_{neg}(z_1)} = 0.604$

$k_{\sigma.4.2b}(\psi_{so}) = 0.613$ Higher stress close to support

$$Class4 := \begin{cases} \text{"Class4"} & \text{if } \frac{h_{so}}{t_{so}} > 21 \cdot \varepsilon(t_{so}) \cdot \sqrt{k_{\sigma.4.2b}(\psi_{so})} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Lower than class 4"}$$

$\lambda_{p.so} := \lambda_p\left(k_{\sigma.4.2b}\left(\psi_{so}\right), h_{so}, t_{so}\right) = 0.484$

$\rho_{d.so} := \rho_{4.3}\left(\lambda_{p.so}, \psi_s\right) = 1$ Internal cross-sectional parts, EC1993-1-5 4.4 (2)

$h_{so.eff} := \rho_{d.so} \cdot h_{so} = 140 \cdot mm$

$A_{so.eff} := n_{so} \cdot t_{so} \cdot h_{so.eff} = 2.016 \times 10^{4} \cdot mm^2$

Deck

$\frac{s_{so}}{t_d} = 35 \qquad 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} = 34.172$

$Class4_ := \begin{cases} \text{"Class4"} & \text{if } \frac{s_{so}}{t_d} > 42 \cdot \sqrt{\frac{235MPa}{f_y\left(t_d\right)}} \\ \text{"Lower than class 4"} & \text{otherwise} \end{cases} = \text{"Class4"}$

$z_d := \frac{-t_d}{2} = -5 \times 10^{-3} m$ $\psi_d := \psi_{neg}\left(z_d, z_d\right) = 1$

$\delta_{s.so} := s_{so} - t_{so}$ [tab. 5.2 ES1993-1-1] $k_{\sigma.d} := k_{\sigma.4.1}\left(\psi_d\right) = 4$

$\lambda_{p.d} := \lambda_p\left(k_{\sigma.d}, s_{c.so}, t_d\right) = 0.723$

$\rho_d := \rho_{4.2}\left(\lambda_{p.d}, \psi_d\right) = 0.962$ Internal cross-sectional parts

$s_{so.eff} := \rho_d \cdot s_{c.so} = 321.461 \cdot mm$

Areas:

$A_{s.so} := n_{so} s_{so} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.31 \times 10^{4} \cdot mm^2$

$A_{c.eff.loc} := n_{so} \cdot s_{so.eff} \cdot t_d + n_{so} \cdot t_{so} \cdot \left(h_{so.eff} + t_d\right) = 5.053 \times 10^{4} \cdot mm^2$

More than three longitudinal stiffeners --> $k_{\sigma.p}$ obtained with eq. A.2

$I_p := \frac{b_d \cdot t_d^{3}}{12 \cdot \left(1 - \nu^2\right)} = 3.205 \times 10^{5} \cdot mm^4$

$y_{tp.sl} := \frac{n_{so} t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + b_d \cdot t_d \cdot \frac{t_d}{2}}{n_{so} t_{so} \cdot h_{so} + b_d \cdot t_d} = 32.411 \cdot mm$

$I_{sl} := \left[\frac{b_d \cdot t_d^{3}}{12} + b_d \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl}\right)^2\right] ... + \left[\frac{n_{so} \cdot t_{so} \cdot h_{so}^{3}}{12} + n_{so} \cdot h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)^2\right] = 1.052 \times 10^{8} \cdot mm^4$

$\gamma := \frac{I_{sl}}{I_p} = 328.143 \quad \alpha := \frac{a}{b_d} = 0.971$

$\psi := 1$ Uniform compression

$A_{sl} := n_{so} \cdot h_{so} \cdot t_{so} = 0.02 m^2$ EC1993-1-5 A.1 (2)

$A_p := b_d \cdot t_d = 0.035 m^2$

$\delta := \frac{A_{sl}}{A_p} = 0.576$

$k_{\sigma.p} := \begin{cases} \frac{2 \cdot \left[\left(1 + \alpha^2\right)^2 + \gamma - 1\right]}{\alpha^2 \cdot (\psi + 1) \cdot (1 + \delta)} & \text{if } \alpha \le \sqrt[4]{\gamma} \\ \frac{4 \cdot \left(1 + \sqrt{\gamma}\right)}{(\psi + 1) \cdot (1 + \delta)} & \text{otherwise} \end{cases} = 222.508$

Critical buckling stress, orthotropic plate:

$\sigma_{cr.sup} := k_{\sigma.p} \cdot \frac{E \cdot \pi^2 \cdot t_d^{2}}{12 \cdot \left(1 - \nu^2\right) \cdot b_d^{2}} = 344.752 \cdot MPa$ EC1993-1-5 eq A.1

$\beta_{A.c} := \frac{A_{c.eff.loc}}{A_c} = 0.952$

$\lambda_{p.sup} := \sqrt{\frac{\beta_{A.c} \cdot f_y\left(t_d\right)}{\sigma_{cr.p}}} = 0.99$

$\rho := \rho_{4.2}\left(\lambda_p, \psi\right) = 0.786$ Plate like behaviour

Effective area of the stiffened plate(deck):

$A_{c.eff} := \rho_{4.2}\left(\lambda_p, \psi\right) \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d = 4.292 \times 10^{4} \cdot mm^2$

Buckling strength

$N_{Rd} := A_{c.eff} \cdot \frac{f_y\left(t_d\right)}{\gamma_M} = 1.524 \times 10^{4} \cdot kN$

Column like behaviour

$y_{tp.sl.c} := \frac{t_{so} \cdot h_{so} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so} \cdot t_d \cdot \frac{t_d}{2}}{t_{so} \cdot h_{so} + s_{so} \cdot t_d} = 34.268 \cdot mm$

$I_{sl.c} := \left[\frac{s_{so} \cdot t_d^{3}}{12} + s_{so} \cdot t_d \cdot \left(\frac{t_d}{2} - y_{tp.sl.c}\right)^2\right] ... + \left[\frac{t_{so} \cdot h_{so}^{3}}{12} + h_{so} \cdot t_{so} \cdot \left(t_d + \frac{h_{so}}{2} - y_{tp.sl.c}\right)^2\right] = 1.137 \times 10^{7} \cdot mm^4$

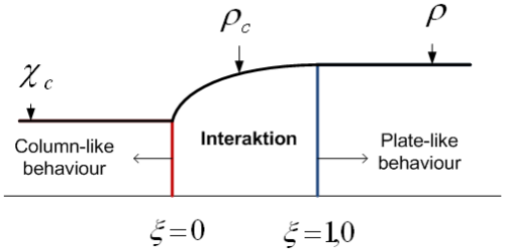
Buckling resistance stiffener

$A_{s.sl} := t_{so} \cdot h_{so} + s_{so} \cdot t_d = 5.74 \times 10^{3} \cdot mm^2$

$A_{sl.eff} := t_{so} \cdot h_{so.eff} + s_{so.eff} \cdot t_d = 5.455 \times 10^{3} \cdot mm^2$

$\beta_c := \frac{A_{sl.eff}}{A_{sl}} = 0.95$

Critical buckling stress for a stiffener



-The stiffener is acting as a column between stiffeners

$\sigma_{cr.sl} := \frac{\pi^2 \cdot E \cdot I_{sl.c}}{A_{sl} \cdot a^2} = 355.172 \cdot MPa$

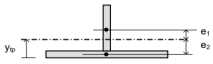
Slenderness ratio

$\lambda_c := \sqrt{\frac{\beta_c \cdot f_y(t_{so})}{\sigma_{cr.sl}}} = 0.975$

Calculation of reduction factor EC1993-1-5 4.5.3

$\alpha := 0.49$ Stiffeners open cross-sections

$e = 2.718$



$i := \sqrt{\frac{I_{sl}}{A_{sl}}} = 135.362 \cdot mm$

$e := \max\left[\left(y_{tp.sl} - \frac{t_d}{2}\right), \left(t_d + \frac{h_{so}}{2} - y_{tp.sl}\right)\right] = 47.589 \cdot mm$

$\alpha_e := \alpha + \frac{0.09}{\frac{i}{e}} = 0.522$

$\phi := 0.5 \cdot \left[1 + \alpha_e \cdot (\lambda_c - 0.2) + \lambda_c^2\right] = 1.177$ EC1993-1-1 6.3.1.2 (1)

$\chi_c := \min\left(1, \frac{1}{\phi + \sqrt{\phi^2 - \lambda_c^2}}\right) = 0.544$

Interaction

$\xi := \max\left(\min\left(\frac{\sigma_{cr.p}}{\sigma_{cr.sl}} - 1, 1\right), 0\right) = 0$

ξ är nästan noll --> column like behviour

Interaction between the two types of behaviour (column/plate buckling)

$\rho_c := (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.544$

Checks

Mean stress rod

The "rod" in this case consist of two parts in the stiffener and one part in the web.

$F_{avst} := n_{so} \cdot s_{so.eff} \cdot t_d \cdot \sigma_{neg}\left(\frac{t_d}{2}\right) + A_{so} \cdot \sigma_{neg}\left(t_d + \frac{h_{so.eff}}{2}\right) = -1.571 \times 10^4 \cdot kN$

$\sigma_{com.Ed} := \frac{F_{avst}}{n_{so} \cdot s_{so.eff} \cdot t_d + A_{so}} = -319.999 \cdot MPa$

Check according to A.2.1 (4)

$\frac{\rho_c \cdot f_y(t_w)}{\gamma_M} = 193.275 \cdot MPa$

$K_{A.2.1} := \begin{cases} \text{"OK"} & \text{if } \frac{\rho_c \cdot f_y(t_b)}{\gamma_M} > \sigma_{com.Ed} \\ \text{"EJ OK"} & \text{otherwise} \end{cases} = \text{"OK"}$

If this condition is not fulfilled, further reductions will be calculated according to A.2.1.(4)

Effective areas, new centre of gravity and moment of inetria

$A_{d.eff} := n_{s.so} \cdot t_d \cdot s_{so.eff} = 0.032\,m^2$

$A_{tot.eff} := A_{so.eff} + A_{d.eff} + A_w + A_{su} + A_b + A_{s.eff} = 0.118\,m^2$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &A_{d.eff} \cdot \frac{t_d}{2} + A_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2}\right) \dots \\ &+ 2 \cdot \left[\begin{aligned} &t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \end{aligned}\right] \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{tot.eff}} = 371.724 \cdot mm$$

$$I_{y.s.eff} := 2 \cdot \left[\begin{aligned} &\frac{t_s \cdot h_{s.eff.c1}^3}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.eff.c2}^3}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \dots \\ &+ \frac{t_s \cdot h_{s.t}^3}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{aligned}\right] = 3.788 \times 10^{-4}\,m^4$$

$I_{y.d.eff} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^3}{12} + n_{s.so} \cdot t_d \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 4.323 \times 10^{-3}\,m^4$

$I_{y.eff.neg} := I_{y.st.so} + I_{y.st.su} + I_{y.w} + I_{y.b} + I_{y.s} + I_{y.s.eff} + I_{y.d.eff} = 0.015\,m^4$

Negative moment

Effective cross section with negative moment(lower flange in tension)

Area

$A_{öf.eff.neg} := A_{d.eff}$

$A_{c.eff.loc} = 5.053 \times 10^4 \cdot mm^2$

$A_{c.eff.loc.mod} := \frac{\rho_c \cdot f_y(t_b) \cdot A_{sl}}{\sigma_{com.Ed} \cdot \gamma_M} = -3.467 \times 10^3 \cdot mm^2$

$A_{uf.eff.neg} := A_b$ Bottom flange in tension

The effective area in compression is calculated according to 4.5.1(4.5). If the condition A.2.1(4) which was checked before is not fulfilled, further reduction is needed and the area above is used.

$A_{c.eff} := \begin{cases} \rho_c \cdot A_{c.eff.loc.mod} + s_{so.eff} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot A_{c.eff.loc} + s_{so.eff} \cdot t_d & \text{otherwise} \end{cases} = 307.258 \cdot cm^2$

A_{su} Lower stiffener in tension -> No reduction.

A_b Tension -> No reduction.

A_s No reduction if the side plate is in class 4.

A_{so} Stiffener class 4 --> No reduction.

Effective areas, effective centre of gravity and effective moment of inertia

$A_{eff.tot} := A_{c.eff} + A_{s.eff} + A_w + A_{su} + A_b = 968.666 \cdot cm^2$

According to SS-EN 1993-1-5 4.5.1 (7), the reduction of the compressive area can be reduced evenly distributed over the cross section. Therefore the web-thickness is changed with earlier area reduction factors.

$t_{d.eff} := \begin{cases} \rho_c \cdot \frac{A_{c.eff.loc.mod}}{A_{c.eff.loc}} \cdot t_d & \text{if } K_{A.2.1} = \text{"EJ OK"} \\ \rho_c \cdot t_d \end{cases} = 5.444 \cdot mm$

$$z_{tp.eff.neg} := \frac{\begin{aligned} &n_{so} \cdot h_{so.eff} \cdot t_{so} \cdot \frac{t_{d.eff}}{t_d} \cdot \left(t_d + \frac{h_{so}}{2}\right) + s_{so.eff} \cdot t_d \cdot \frac{t_d}{2} \dots \\ &+ n_{so} \cdot s_{so.eff} \cdot t_{d.eff} \cdot \left(\frac{t_d}{2}\right) + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2}\right) \dots \\ &+ t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2}\right) + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2}\right) \dots \\ &+ A_w \cdot \left(t_d + h_s + \frac{h_{wy}}{2}\right) + A_{su} \cdot \left[t_d + h_s + \frac{2}{3}(h_{wy} - s_{su0}) - \frac{h_{su.y}}{2}\right] \dots \\ &+ A_b \cdot \left(h_{ts} - \frac{t_b}{2}\right) \end{aligned}}{A_{eff.tot}} = 436.408 \cdot mm$$

Effective moment of inertia:

$$I_{y.d.eff.fin} := \frac{n_{s.so} \cdot s_{so.eff} \cdot t_d^{3}}{12} + t_d \cdot n_{s.so} \cdot s_{so.eff} \cdot \left(\frac{t_d}{2} - z_{tp.eff.neg}\right)^2 = 5.983 \times 10^{-3}\,m^4$$

$$s_{su0y} := \sin\left(21.12 \cdot \frac{\pi}{180}\right) s_{su0} = 7.206 \cdot mm$$

Stiffeners over/under:

$$I_{y.st.so.fin} := n_{so} \cdot \frac{t_{so} \cdot h_{so.eff}^{3}}{12} + n_{so} \cdot t_{so} \cdot h_{so.eff} \cdot \left(t_d + \frac{h_{so.eff}}{2} - z_{tp.eff.neg}\right)^2 = 2.594 \times 10^{-3}\,m^4$$

$$I_{y.st.su.1.fin} := 6 \cdot 1.506 \cdot 10^{-6}\,m^4 \quad \text{from FEM}$$

$$I_{y.st.su.fin} := I_{y.st.su.1.fin} \cdots = 5.869 \times 10^{-4}\,m^4$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{1}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \frac{2}{3}\left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2 \cdots$$
$$+ 2t_{su} \cdot h_{su} \cdot \left[t_d + h_s + \left(h_{wy} - s_{su0y}\right) - \frac{h_{su.y}}{2} - z_{tp.eff.neg}\right]^2$$

Web:

$$I_{y.w.1.fin} := 2 \cdot 0.000312\,m^4 \quad \text{from FEM}$$

$$I_{y.w.fin} := I_{y.w.1.fin} + 2t_w \cdot h_w \cdot \left[h_{ts} - \frac{t_b}{2} - \frac{1}{2} \cdot \left(h_{wy}\right) - z_{tp.eff.neg}\right]^2 = 1.965 \times 10^{-3}\,m^4$$

Bottom flange:

$$I_{y.b.fin} := \frac{b_b \cdot t_b^{3}}{12} + t_b \cdot b_b \cdot \left(h_{ts} - z_{tp.eff.neg} - \frac{t_b}{2}\right)^2 = 3.177 \times 10^{-3}\,m^4$$

Side plate:

$$I_{y.s.fin} := 2 \cdot \left[\begin{array}{l} \frac{t_s \cdot h_{s.eff.c1}^{3}}{12} + t_s \cdot h_{s.eff.c1} \cdot \left(t_d + \frac{h_{s.eff.c1}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.eff.c2}^{3}}{12} + t_s \cdot h_{s.eff.c2} \cdot \left(t_d + h_{s.c} - \frac{h_{s.eff.c2}}{2} - z_{tp.eff.neg}\right)^2 \cdots \\ + \frac{t_s \cdot h_{s.t}^{3}}{12} + t_s \cdot h_{s.t} \cdot \left(t_d + h_s - \frac{h_{s.t}}{2} - z_{tp.eff.neg}\right)^2 \end{array}\right] = 6.198 \times 10^{-4}\,m^4$$

$$I_{y.eff.neg.fin} := I_{y.d.eff.fin} + I_{y.st.so.fin} + I_{y.st.su.fin} + I_{y.w.fin} + I_{y.b.fin} + I_{y.s.fin} = 1.493 \times 10^{10} \cdot mm^4$$

Load

$q_M := 85\,\frac{kN}{m^2}$ The load capacity is q_M when checking the moment capacity.

$$Q := q_M \cdot b_d = 297.5 \cdot \frac{kN}{m} \qquad V_{Ed} := \frac{Q \cdot l_{bridge}}{2} = 2.678 \times 10^{3} \cdot kN$$

$$M_{Ed} := \frac{Q \cdot l_{bridge}^{2}}{8} = 1.205 \times 10^{4} \cdot kN \cdot m$$

Moment capacity

$$W_{y.eff} := \frac{I_{y.eff.neg.fin}}{z_{tp.eff.neg}} = 0.034 \cdot m^3$$

$$M_{Rd} := W_{y.eff} \cdot \frac{f_y(t_d)}{\gamma_{M1}} = 1.214 \times 10^{4} \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{Rd}} = 0.992$$

Shear capacity

$\eta := 1.20$ SS-EN 1993-1-5 5.1 ANM 2

Shear buckling coefficients

Moment of inertia longitudinal stiffeers

According to figure 5.3 SS-EN1993-1-5, 15εt on each side of the stiffener is used when calculating the moment of inertia.

$$b_{web} := 2 \cdot 15 \cdot \varepsilon(t_w) \cdot t_w = 0.293\,m$$

$$\alpha_w := \operatorname{asin}\left(\frac{h_{wy}}{h_w}\right) \cdot \frac{180}{\pi} = 21.119 \qquad \text{Angle of web}$$

Distance to stiffeners from z-axis:

$$a_{st.z.1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} - \frac{\sin\left(\alpha_w \cdot \frac{\pi}{180}\right) \cdot h_{su}}{2} = 0.503\,m$$

$$a_{st.z.2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.913\,m$$

$$a_{st.z.3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.324\,m$$

Moment of inirtia stiffeners z-direction

$$I_{st.z} := 2t_{su} \cdot h_{su} \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 8.18 \times 10^{-3}\,m^4$$

Distance from z-axis to middle of the web area which is 15εt each side of the stiffeners:

$$a_{stwz1} := \frac{b_b}{2} + s_{su0} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) + \frac{t_{su}}{2} = 0.525\,m$$

$$a_{stwz2} := a_{st.z.1} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 0.935\,m$$

$$a_{stwz3} := a_{st.z.2} + s_{su} \cdot \cos\left(\alpha_w \cdot \frac{\pi}{180}\right) = 1.346\,m$$

Moment of inirtia of 15εt of the web z-direction

$$I_{stw.z} := 2b_{web} \cdot t_w \cdot \left[\left(a_{st.z.1}\right)^2 + \left(a_{st.z.2}\right)^2 + \left(a_{st.z.3}\right)^2\right] = 0.021 \cdot m^4$$

$$I_{st.z.tot} := I_{st.z} + I_{stw.z} = 0.029\,m^4$$

Shear buckling coefficient, side plate

$$k_{\tau 1} := \begin{cases} 5.34 + 4.00 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} \geq 1 \\ 4.00 + 5.34 \cdot \left(\frac{h_{s.eff}}{a}\right)^2 & \text{if } \frac{a}{h_{s.eff}} < 1 \end{cases} = 5.392$$

SS-EN 1993-1-5 5.3 (5) ANM. A.3 when $\kappa_{st} = 0$

$$\lambda_{w1} := \frac{h_{s.eff}}{37.4 \cdot t_s \cdot \varepsilon(t_s) \cdot \sqrt{k_{\tau 1}}} = 0.459 \qquad \text{EC1-3-5 eq.5.6}$$

Shear buckling coefficient; web with more than two longitudinal stiffeners

$$\alpha := \frac{a}{h_w} = 2.537$$

SS-EN 1993-1-5 (A.5)

$$k_{\tau sl} := \max\left[9 \cdot \left(\frac{h_w}{a}\right)^2 \cdot \sqrt[4]{\left[\frac{I_{st.z.tot}}{\left(\min\left(t_s, t_w\right)\right)^3 \cdot h_w}\right]^3}, \frac{2.1}{\min\left(t_s, t_w\right)} \cdot \sqrt[3]{\frac{I_{st.z.tot}}{h_w}}\right] = 1.655 \times 10^{3}$$

SS-EN 1993-1-5 A.3 (1) and (2) gives:

$$k_\tau := \begin{cases} 5.34 + 4\left(\frac{h_{web}}{a}\right)^2 + k_{\tau sl} & \text{if } \alpha \geq 3 \\ 4.1 + \frac{6.3 + 0.18 \cdot \frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}}{\alpha^2} + 2.2 \sqrt[3]{\frac{I_{st.z.tot}}{\min\left(t_s, t_w\right)^3 \cdot h_{web}}} & \text{otherwise} \end{cases} = 607.09$$

Slenderness limit for which shear buckling check has to be performed for stiffened webs:

$\eta := 1.2$ EC1993-1-3-5 ANM.2 5.1

$$Check := \begin{cases} \text{"no shear buckling"} & \text{if } \frac{h_{web}}{\min(t_s, t_w)} \le 31 \cdot \frac{\varepsilon(t_s)}{\eta} \cdot \sqrt{k_\tau} \\ \text{"Reduce"} & \text{otherwise} \end{cases} = \text{"Reduce"}$$

Reduce --> Shear buckling check is needed, EC1003-1-5 Ch. 5.1(2).

$$\lambda_w := \max\left(\frac{h_w}{37.4 t_w \cdot \varepsilon(t_w)\sqrt{k_\tau}}, \lambda_{w1}\right) = 0.459$$ SS-EN 1993-1-5 5.3 (5.6)

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_w < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_w} & \text{if } \frac{0.83}{\eta} \le \lambda_w < 1.08 \\ \frac{1.37}{0.7 + \lambda_w} & \text{if } \lambda_w \ge 1.08 \end{cases} = 1.2$$ SS-EN 1993-1-5 5.3 Table 5.1, stiff endstiffener.

Kapacity of the web

$\gamma_{M1} := 1.00$ $\gamma_{Mw} := 1.2$ Safety class 3

$\gamma_{M0} := 1.00$

Shear capacity:

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1} \cdot \gamma_n} = 2.125 \times 10^3 \cdot kN$$ SS-EN 1993-1-3 5.2 (5.2)

$V_{b.Rd} := V_{bw.Rd}$

$V_{Ed} = 2.678 \times 10^3 \cdot kN$

$$\frac{V_{Ed}}{2V_{bw.Rd}} = 0.63$$

$$V_{b.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$

Kapacity of the flanges

SS-EN 19931-5 5.4

No shear buckling --> the plastic load bearing capacity can be calculated.

Moment of inertia

$a_{öf.eff.neg} := z_{tp.eff.neg} - \frac{t_d}{2} = 431.408 \cdot mm$

$a_{uf.eff.neg} := h_{ts} - \frac{t_b}{2} - z_{tp.eff.neg} = 445.592 \cdot mm$

The flanges plastic moment bearing capacity for a cross-section with the effective flange-area.

$$M_{f.Rd.neg} := \frac{A_{öf.eff.neg} \cdot |a_{öf.eff.neg}| \cdot f_y(t_d)}{\gamma_{M0}} + \frac{A_{uf.eff.neg} \cdot |a_{uf.eff.neg}| \cdot f_y(t_b)}{\gamma_{M0}} = 7.454 \times 10^3 \cdot kN \cdot m$$

The contribution from the flanges should be taken into account when there are still capacity left in the flanges in bending, ie the following condition should be fullfilled:

$M_{Ed} < M_{f.Rd.neg} = 0$

$$c_{öf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_d \cdot t_d^2 \cdot f_y(t_d)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 1.059\,m$$

$$c_{uf} := a \cdot \left[0.25 + \frac{1.6 \cdot b_b \cdot t_b^2 \cdot f_y(t_b)}{t_s \cdot (h_s + h_{wy})^2 \cdot f_y(t_s)}\right] = 1.003\,m$$

$$V_{bf.Rd.ored} := \frac{b_d \cdot t_d^2 \cdot f_y(t_d)}{c_{öf} \cdot \gamma_{M1} \cdot \gamma_n} + \frac{b_b \cdot t_b^2 \cdot f_y(t_b)}{c_{uf} \cdot \gamma_{M1} \cdot \gamma_n} = 173.339 \cdot kN$$

The flanges contribution according to EC1993-1-5 Ch. 5.4(1) eq.5.8

$$V_{bf.Rd} := \begin{cases} V_{bf.Rd.ored} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f.Rd.neg}}\right)^2\right] & \text{if } M_{Ed} < M_{f.Rd.neg} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot kN$$

Shear capacity stiffened web:

$$V_{bw.Rd} + V_{bf.Rd} \le \frac{\eta \cdot f_y(t_s) \cdot h_{web} \cdot t_s}{\sqrt{3} \cdot \gamma_{M1}} = 1$$ EC1993-1-5 Ch.5.2(1)

$V_{bw.Rd} := V_{bw.Rd} + V_{bf.Rd} = 2.125 \times 10^3 \cdot kN$

Check:

$$\frac{\frac{V_{Ed}}{2}}{V_{b.Rd}} \le 1 = 1$$ EC1993-1-5, Ch.5.5, eq. 5.10

$$\frac{V_{Ed}}{2V_{b.Rd}} = 0.63$$

Flanges, plastic moment bearing capacity for the effective-flange-area.

Moment of inertia of the flanges:

$I_{y.f.eff.fin} := I_{y.d.eff.fin} + I_{y.b.fin} = 9.16 \times 10^{-3} \cdot m^4$

The flanges effective area:

$A_{eff.f.tot} := s_{so.eff} \cdot t_d + n_{so} \cdot s_{so.eff} \cdot t_{d.eff} + n_{so} \cdot t_{so} \cdot t_{d.eff} + A_b = 357.5 \cdot cm^2$