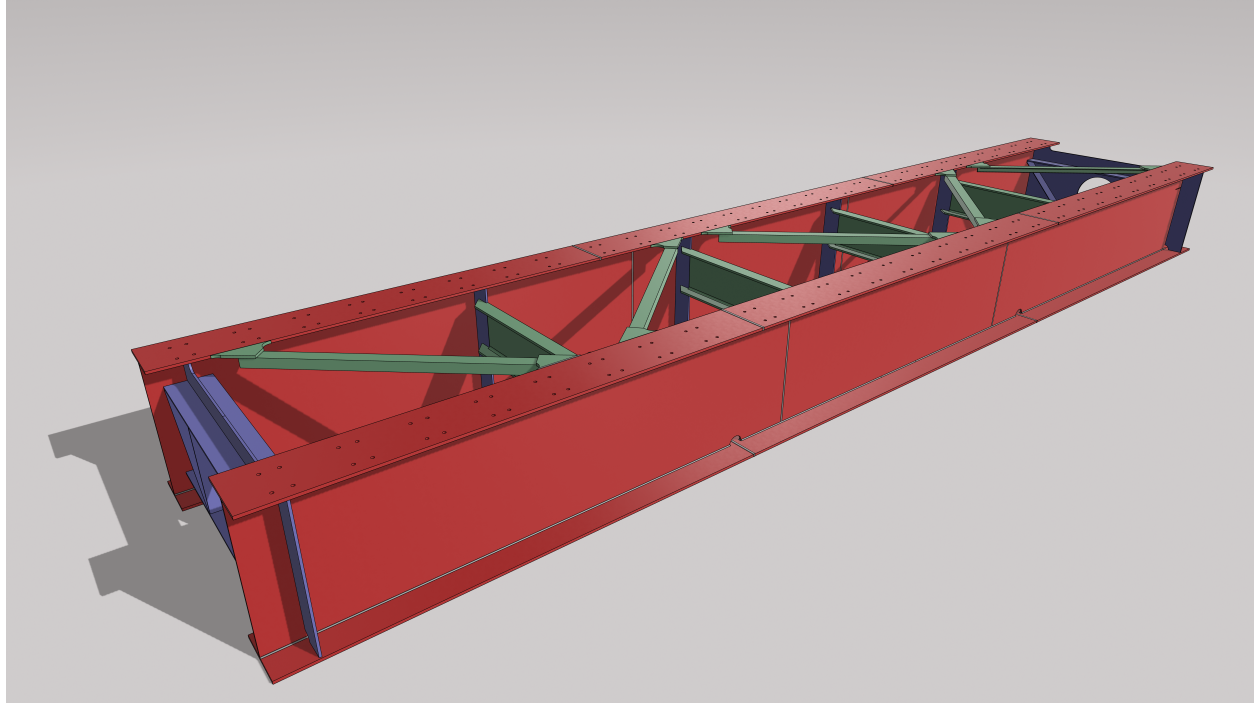




CHALMERS
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Design Optimization of Railway Steel Bridges

Utilizing High Frequency Mechanical Impact Treatment

Master's Thesis Structural Engineering and Building Technology

EMIN GUSTAFSSON
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DEPARTMENT OF ARCHITECTURE AND CIVIL ENGINEERING

CHALMERS UNIVERSITY OF TECHNOLOGY

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Cover: Overview of the reference bridge used in this study, a twin I-girder with stiffener plates.

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Abstract

Railway bridges are highly susceptible to fatigue, which typically governs their design. By applying High Frequency Mechanical Impact (HFMI) treatment to enhance fatigue strength, new opportunities arise to effectively utilize Higher Strength Steel (HSS) without increasing dimensions for the cross-section. To evaluate this potential, a Parametric Design Model (PDM) was developed in MATLAB and combined with a Genetic Algorithm (GA) to optimize simply supported, single-track, twin I-girder railway bridges (10–40 m spans) across steel grades S355, S460, and S690. The GA optimizes the cross-sectional geometry to minimize total initial production costs taking encompassing material, welding, and painting in to account while strictly verifying the designs against Fatigue Limit State (FLS), Ultimate Limit State (ULS), and Serviceability Limit State (SLS) criteria.

This thesis then investigates whether it is profitable, and in which combinations of HFMI and HSS, the potential for material and cost savings is greatest. The results demonstrate that the most significant cost savings are achieved using HFMI in combination with steel grade S355. While steel grades S460 and S690 provide some benefits compared to the As-Welded (AW) case for shorter span lengths of 10 m, its economic viability is severely restricted for longer spans. A key finding regarding the failure modes is that while FLS strictly governs the AW that is used as the reference designs case, the application of HFMI shifts the governing failure mode to ULS for shorter spans, and entirely to SLS deflection for spans over 20 m.

To get representative results the optimization was done considering total investment cost. Conversely, data was analysed comparing both impact on CO_2 -emissions and total Life Cycle Cost (LCC). These comparisons implied some even greater potential for design optimization utilizing HFMI if full life cycle were to be considered. In addition, an easier comparisons of fatigue design methods was made, which indicated that the positive benefits from HFMI is method independent.

Keywords: HFMI, Fatigue, Railway bridges, Design optimization, Overload effects, Genetic algorithm, HSS, ESR, λ_{HFMI} .

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Emin Gustafsson and Eric Nilsson, Gothenburg, June 2026

List of Acronyms

AI	Artificial Intelligence
AW	As-welded
BG	Burr Grinding
DAS _t	German Committee for Steel Construction
ESR	Equivalent Stress Range
FAT	Fatigue Class (referring to the characteristic fatigue strength at $2 \cdot 10^6$ cycles)
FEA	Finite Element Analyses
FEM	Finite Element Method
FLS	Fatigue Limit State
GA	Genetic Algorithm
HFMI	High Frequency Mechanical Impact
HSS	High-Strength Steel
IIW	International Institute of Welding
LCC	Life Cycle Cost
LM71	Load Model 71
PWT	Post Weld Treatment
SLS	Serviceability Limit State
TIG	Tungsten Inert Gas (referring to TIG-dressing)
ULS	Ultimate Limit State
PDM	Parametric Design Model

Nomenclature

Below is the nomenclature of indices, parameters and variables that have been used throughout this thesis.

Indices

i, j Indices for nodes, units, or specific stress cycles

Parameters

$A_{ref,x}$	Reference area of the structure for wind actions
C_{fx}	Shape factor of the structure for wind load calculations
$\alpha_{traffic}$	Load magnification factor for Load Model 71
β_{rs}	Rail shift factor to consider track eccentricity
γ_{Mf}	Partial factor for fatigue resistance
γ_{Ff}	Partial factor for applied stress ranges $\Delta\sigma_E$
$\gamma_{G,self}$	Partial factor for permanent action (self-weight)
$\gamma_{Q,LM71}$	Partial factor for variable action (Load Model 71)
$\gamma_{Q,nosing}$	Partial factor for variable action (nosing forces)
$\gamma_{Q,traction}$	Partial factor for variable action (traction forces)
$\gamma_{Q,wind}$	Partial factor for variable action (wind loads)
f_1	Factor for steel yield strength on fatigue strength of HFMI-treated welds
f_2	Factor for the effect of R-ratio on fatigue strength of HFMI-treated welds
k_s	Reduction factor for fatigue resistance to account for size effects
k_f	Stress concentration factor for thickness taper
m_1, m_2	Slope parameters of a fatigue resistance curve
L_Φ	Determining length of the structural element

$\Delta\sigma_c$	Characteristic fatigue resistance in as-welded condition at $2 \cdot 10^6$ cycles
$\Delta\sigma_{C,HFMI}$	Characteristic fatigue resistance for HFMI-treated detail at $2 \cdot 10^6$ cycles
$\Delta\sigma_{C,HFMI,ref}$	Reference value for fatigue resistance of HFMI-treated detail ($f_y = 355$ MPa, $R = 0.1$)
$\Delta\sigma_D$	Constant amplitude fatigue limit (knee point)
$\Delta\sigma_L$	Cut-off limit
$\Delta\sigma_{D,HFMI}$	Characteristic constant amplitude fatigue limit (knee point) for HFMI-treated detail
$\Delta\sigma_{D,HFMI,ref}$	Reference value for constant amplitude fatigue limit of HFMI-treated detail
$\Delta\sigma_{LM71}$	Maximum stress range generated from Load Model 71 (used for calculating Φ)
$\Delta\sigma_s$	Characteristic stress range at intersection of fatigue resistance curves in as-welded and HFMI-treated state
σ_{perm}	Stress from permanent loads
λ_{HFMI}	Damage equivalent factor to account for mean stress effect in spectrum loading
$\lambda_1, \lambda_2, \lambda_3, \lambda_4$	Damage equivalent factors in the simplified λ -coefficient method
Φ	Ratio of permanent stress to stress range of the load model
Φ_2	Dynamic factor for railway traffic loads
Q_{vk}	Characteristic vertical point load for Load Model 71
q_{vk}	Characteristic vertical distributed load for Load Model 71
q_{empty}	Distributed load representing empty wagons
q_p	Characteristic peak velocity pressure
v_b	Basic wind velocity

Variables

D	Accumulated fatigue damage due to different stress ranges ($D = \sum D_i$)
R	The ratio of minimum to maximum stress in a stress cycle
$C_{material}$	Total material cost
C_{paint}	Total cost of the surface treatment
C_{total}	Total structural cost (objective function for optimization)
C_{weld}	Total welding cost
c_{weld}	Unit cost of welding per meter
n_{passes}	Number of welding passes required for a specific weld cross-section
$\Delta\sigma_E$	Applied fatigue stress range
$\Delta\sigma_{Ed}$	Fatigue action design effect (stress ranges)
$\Delta\sigma_{e,2,HFMI,Ed}$	Equivalent design stress range for HFMI-treated joints
$\Delta\sigma_{eq}$	Equivalent stress range for a variable amplitude loading
$\Delta\sigma_p$	Principal stress range
$\Delta\tau$	Shear stress range
$F_{w,x}$	Characteristic transverse wind force
N_{eq}	Equivalent number of cycles from variable amplitude loading
$N_{min,HFMI}$	Minimum number of cycles to failure for HFMI-treated details
σ_{max}	Maximum stress of the characteristic load combination in SLS
σ_{min}	Minimum stress from non-permanent load
$Q_{h,dim}$	Longitudinal design force
Q_{lak}	Characteristic traction force
Q_{lbk}	Characteristic braking force

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1

Introduction

This chapter defines the aim, objectives, and limitations of the study, and provides the necessary background for the thesis. It introduces the role of High Frequency Mechanical Impact (HFMI) treatment as a tool to optimize the design of railway steel bridges, specifically by enabling higher material efficiency and extending service life. A twin I-girder railway bridge is evaluated across various spans to determine the conditions under which HFMI treatment provides the most significant structural and economic benefits.

1.1 Background

Bridges play a central role in public infrastructure and transport systems and are important assets for the national economy. Therefore, effective maintenance and repair strategies are crucial not only to minimise operational disruptions but also to control the significant costs associated with infrastructure management. Unexpected closures due to structural defects have serious economic consequences, including lost revenue for freight operators and high direct costs for emergency repairs. An example of this is the 2023 disruption of the Iron Ore Line (Malmbanan), where LKAB claimed losses of 100 million SEK for every day trains were out of operation (Sternlund, 2023). Ensuring the reliability of these structures is therefore both a logistical necessity and a matter of economic efficiency.

In Sweden, Trafikverket manages approximately 21,500 bridges, of which 4,500 are steel railway structures (Trafikverket, 2025). These bridges are frequently subjected to cyclical loading, which can lead to accumulation of damage over time, even at stress levels well below yield limit of the steel (Al-Emrani & Åkesson, 2020). This phenomenon is known as fatigue.

In bridge design, the fatigue life of welded details is frequently the governing limit state (Al-Emrani & Aygöl, 2014). When fatigue strength cannot be improved, designers are forced to use heavier and more expensive sections to meet service life requirements. This leads to increased material consumption and greater environmental impact, highlighting the need for effective fatigue enhancement methods such as HFMI. Post Weld Treatment (PWT) methods have been proven to be a promising approach to address these fatigue challenges, which encompass various techniques designed to enhance the local fatigue resistance of welded joints. Among these, HFMI has emerged as a highly effective alternative. HFMI improves fatigue

strength primarily by modifying the local weld geometry and introducing beneficial compressive residual stresses at the weld toe (Al-Emrani et al., 2024; Marquis & Barsoum, 2016). By mitigating the impact of surface weld defects and stress concentrations, HFMI allows more efficient utilization of steel structures.

In contrast to untreated (as-welded) details, the fatigue strength of HFMI-treated joints is highly dependent on factors such as steel grade and specific loading conditions. It is of significant interest to investigate the possible gains of using Higher Strength Steel (HSS) in combination with HFMI. Especially since the second generation of Eurocode introduces allowance of design with steel grades up to S700 (Swedish Standards Institute, 2022). Although HFMI offers significant potential for optimization, its effectiveness can be limited by high stress ratios (R-ratio) or peak loads that can damage the positive effects of details treated with HFMI (Al-Emrani et al., 2024). Despite these considerations, earlier research has demonstrated that HFMI significantly improves the material efficiency of road bridges (Englund & Dagduk, 2022). Consequently, there is a strong incentive to investigate whether similar benefits can be achieved for railway bridges, which involve additional factors that need consideration. Both loading patterns differ and the risk of lateral torsional buckling (LT-buckling) becomes a restrictive factor, due to the absence of restraint for the compressed flange, for example a bridge deck of a composite bridge, such as a road bridge.

In this study, the type bridge will be based on an existing bridge in Sweden. The bridge in question is a double I-girder bridge with two simply supported beams as the main load carrying structure, see Figure 1.1. A study conducted by Hofstetter et al., 2026 has shown I-girders to be the most materially efficient alternative for single-track, simply supported railway bridges. This confirms that the reference bridge used in this work is a highly relevant and efficient choice given the requirements studied in this thesis. The stiffening system of the main girders consists of a truss system connected to the top flange and the web of the main girders. The transverse elements in the truss system are UPE-beams and the diagonals L-profiles. Transverse bulkheads are also added to the supports. The initial design parameters are further detailed in Chapter 3.

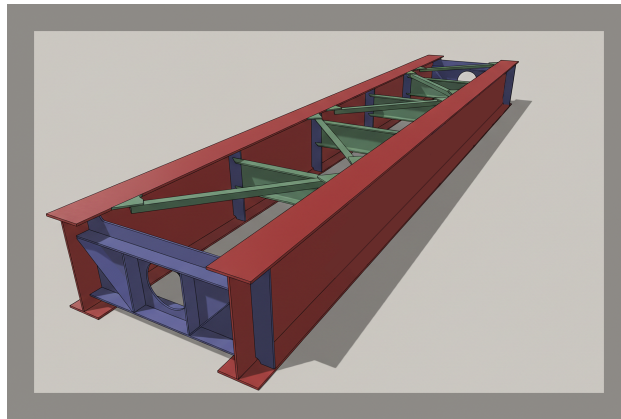


Figure 1.1: Principal model of the railway bridge type.

As mentioned earlier, the thesis will investigate HFMI treatment as a tool to optimize railway bridges. The optimization of the algorithm design performed in this work will be conducted using a Genetic Algorithm (GA) (MathWorks, n.d.). GA has earlier been successfully applied to structural engineering problems in similar studies, for instance, to optimize material usage in high-speed railway bridges while accounting for all relevant limit states (Hofstetter et al., 2026). GA is a method of optimization based on natural evolution theory, a selection process where the most adapted solutions will be propagated in the optimization process (Kärvegård et al., 2025). The method is a stochastic method which means that it will probably not generate the exact same solution several times over, this is a result of the process conducted by the GA. The stages of a GA include the generation of candidate solutions, the evaluation of how they comply with the constraints and their fitness, the selection of the most adapted individuals, and the creation of a new population through mechanisms such as crossover and mutation among the best solutions. This iterative process is repeated until a satisfactory solution is found or a predefined termination criterion is met.

1.2 Aim

The aim of this thesis is to investigate the potential of HFMI treatment to improve the material efficiency and service life of steel railway bridges. Based on the established fact that HFMI increases fatigue strength in critical welded details, this study investigates the implications of its application on the structural design of steel railway bridges. In this study a double I-girder bridge is based on an existing bridge in Sweden. The investigation includes a parametric study that considers variables such as span lengths, steel grades, various cross-sectional dimensions and also comparing HFMI-treated with untreated design.

1.3 Objective

To meet the aim of the thesis, the following objectives have been established:

- Develop a **Parametric Design Model (PDM)** to evaluate and optimize railway bridge configurations by integrating HFMI enhancement effects with standard **Eurocode requirements** for:
 - **Fatigue Limit State (FLS)**,
 - **Ultimate Limit State (ULS)**,
 - **Serviceability Limit State (SLS)**.
- Perform a **parametric study** to identify the **optimal combination** of:
 - **Plate thicknesses**,
 - **Steel grades**,
 - **Cross-section geometry**,by exploiting the **enhanced fatigue strength** provided by **HFMI treatment**.
- Quantify **material weight savings** and **cost reductions** by comparing **optimized HFMI-treated designs** with **conventional untreated solutions**.

- Investigate the influence of the following **considered parameters**:
 - **Steel grades**: S355, S460 & S690,
 - **Span lengths**: 10, 20, 30 & 40 m,
 - **Plate thickness**: using available discrete plate thicknesses from Tibnor, n.d. with a maximum thickness of 60 mm,
 - **HFMI treatment condition**: factory-treated and untreated (as-welded),
 - **Lateral support of compressed flange**,
 - **Spacing between transverse stiffeners**,
 - **Weld sizes**: within allowable limits (4-8 mm).
- Define the **GA-optimization objectives** as:
 - **Minimizing costs, including material, surface treatment and manufacturing.**

1.4 Limitations

Given the time constraints, specific limitations have been defined to ensure the feasibility of the study within the scope of the thesis. The limitations are:

- The thesis is restricted to a **railway bridge**, other forms of **bridges will not** be considered.
- Only **one specific twin I-girder railway bridge** will be considered in the study.
- **Limiting parameters**: **One type of bridge structure (double I-girder bridge), one type of stiffening system (a truss system of diagonal L-profiles and UPE-profiles as transverse stiffeners).**
- The study is limited to an evaluation of span lengths in **increments of 10 m** within the 10–40 m range.
- Fatigue loading is modelled using only **standard traffic mix, Load Model 71 (LM71) in cases where λ -method is used for fatigue assessment.**
- Dynamic analysis, effect or design is not considered as a constraint in this study.

1.5 Method

This master's thesis examines the impact that HFMI treatment has on railway bridges by studying a twin I-girder bridge. Furthermore, the thesis aims to identify in which cases HFMI treatment combined with different steel classes is favourable based on material usage for different span lengths of the specific bridge type. This can lead to potential material and cost savings associated with the application of HFMI and further clarify in which cases a specific combination of HFMI and steel class is favourable for different span lengths.

The theoretical framework of this thesis is established through a systematic literature review drawing inspiration from Sandahl, 2021, and is guided by a critical approach to source evaluation. To ensure the inclusion of high-quality peer-reviewed research, Scopus was utilized as a primary search tool. The search strategy used Boolean

operators and advanced filters, supplemented by the Scopus Artificial Intelligence (AI) feature. This feature was used to refine the results and identify specialized publications that might be overlooked in standard keyword searches. Typical search terms included '*Steel bridge*', '*fatigue*', and '*HFMI*'. Beyond the literature review and computational modelling, hand calculations are performed for verification purposes. This verification process is essential to validate the accuracy of the PDM and ensures that optimized designs remain within the safety margins defined by the upcoming Eurocode guidelines.

To provide a structured overview of the research process, the general methodology is illustrated in Figure 1.2. The process is divided into four main phases: **Input & Background**, **Development & Verification**, **Optimization**, **Analysis & Results**. This systematic approach ensures that the transition from theoretical guidelines to parametric implementation is evaluated through continuous comparison with hand calculations according to established design standards.

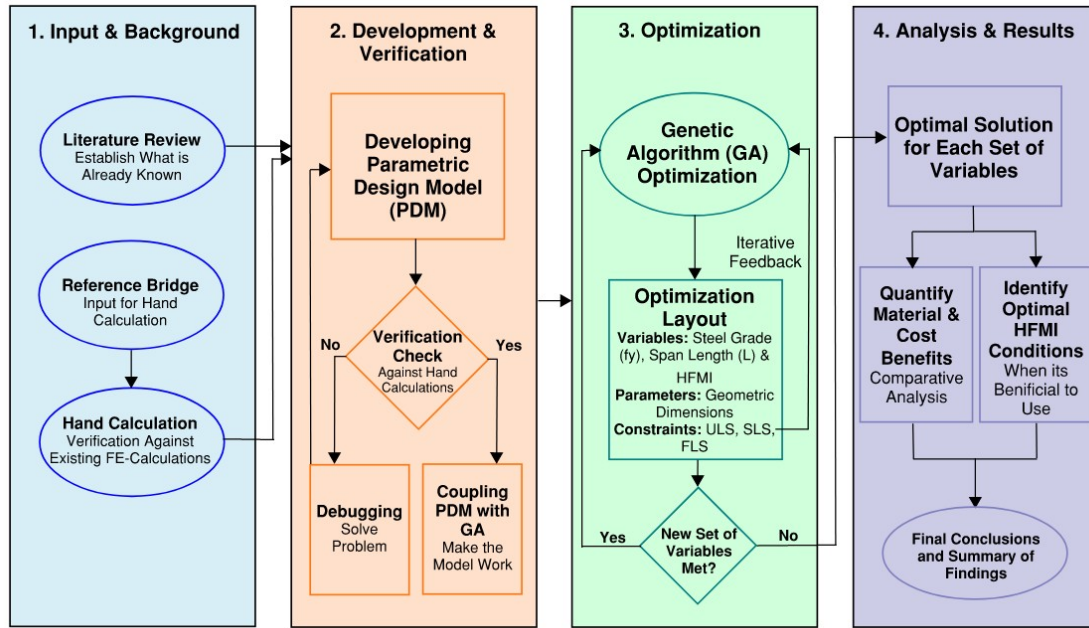


Figure 1.2: Flowchart of the methodology.

The first phase in Figure 1.2 integrates findings from the systematic literature review with structural data from the reference bridge. In the second phase a parametric framework based on Eurocode design approaches is developed. A critical step in this phase is the verification, where the automated results are cross-referenced with the first-hand calculations to ensure structural integrity. The third phase consists of an optimization process implementing a GA (MathWorks, n.d.), with the PDM, where combinations of design variables such as steel grades, span lengths, and with-/without HFMI implemented will be evaluated. In each set of those combinations plate thicknesses and size are iteratively evaluated against Eurocode constraints for ULS, SLS, and FLS. Finally, the results will be analysed to quantify the benefits

of the HFMI treatment in terms of material efficiency and cost reduction. The combinations of variables that will be considered are shown in Figure 1.3.

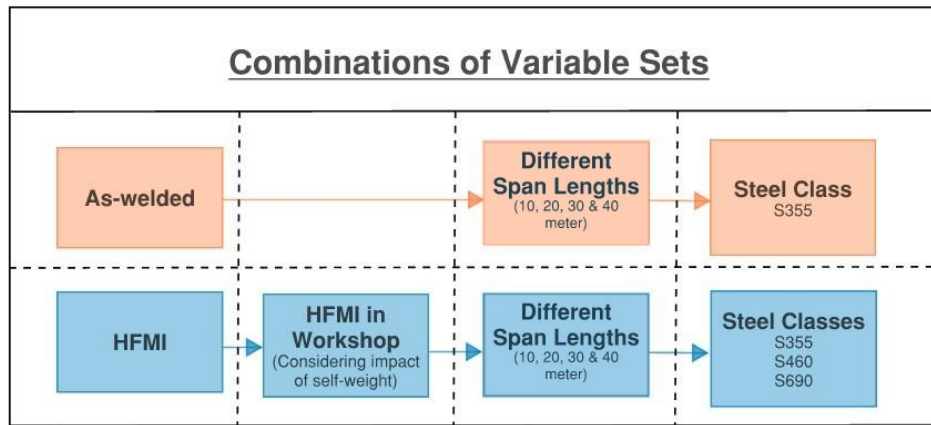


Figure 1.3: Flowchart of sets of variables that will be tested in the optimization study.

2

Theoretical Background

In the following sections, fatigue as a phenomenon is discussed and how the relatively new HFMI technique can be used to improve fatigue strength in welds. As mentioned earlier, fatigue is often one of the most limiting factors in steel structures susceptible to cyclic loading, which is the case for the railway bridge loaded repetitively by trains throughout their service life.

Although the theoretical foundation and general benefits of HFMI are well-documented in the existing literature, there is still a need to quantify its potential benefits in specific design contexts. While previous studies have focused primarily on establishing theoretical knowledge and design standards, there is limited research on the actual implementation in the design of new structures. Therefore, this thesis focuses on the applicability of HFMI for railway bridge design, specifically examining to what extent bridge cross-sections can be optimized to reduce material usage and costs.

2.1 Fatigue

Steel structures such as railway bridges that are susceptible to cyclic loading, even well below yield stress, accumulate small amounts of damage over time and this phenomenon is known as fatigue (Al-Emrani & Åkesson, 2020). Fatigue cracks often initiate from regions with a larger amount of imperfections, in combination with higher stress concentrations. An example of both factors occurring simultaneously is in welded connections, where both the weld profile disturbs the stress flow and the residual stresses, from the welding process, amplify the stress concentration simultaneously. In addition to the high stress concentration, the welding process is also a source of small deformations, such as micro cracks, from which the cracking process easier could initiate.

2.1.1 Fatigue Limit State

In Fatigue design, the so called S-N curves are used to determine fatigue life of different categorised details (Al-Emrani & Åkesson, 2020). The S-N curves and descriptions of the detail categories can be found in a part of the Eurocode standard, *Eurocode 3: Design of steel structures- Part 1-9: Fatigue* (Swedish Standards Institute, 2005c). Those curves represent the number of load cycles plotted against

the stress range in logarithmic scales. The λ -coefficient method could also be used to verify the fatigue life of a welded detail. λ -coefficient method calibrates LM71 with respect to bridge specific factors such as span length divided by the influence line, the annual volume of heavy vehicles, the required design life, and the effects of simultaneous loading from multiple tracks according to *Eurocode 3: Design of steel structures - Part 2: Steel bridges* (Swedish Standards Institute, 2006). The S-N curves describe the relationship between constant amplitude stress ranges and loading cycles. Thus, in the case of variable amplitude loading, a summation method needs to be used. An example of such a method is damage accumulation using the Equivalent Stress Range (ESR).

The S-N curve for an as-welded (AW) detail (not considering shear stresses) has a slope of 1:3 until the knee point (stress range for constant amplitude fatigue limit $\Delta\sigma_D$) at 5 million cycles, followed by a slope of 1:5 until the cut-off limit (stress range $\Delta\sigma_L$) at 100 million stress cycles according to *Eurocode 3: Design of steel structures- Part 1-9: Fatigue* (Swedish Standards Institute, 2005c). The values of the stress range representing the detail category (stress range $\Delta\sigma_C$) are given at 2 million cycles, see Figure 2.1.

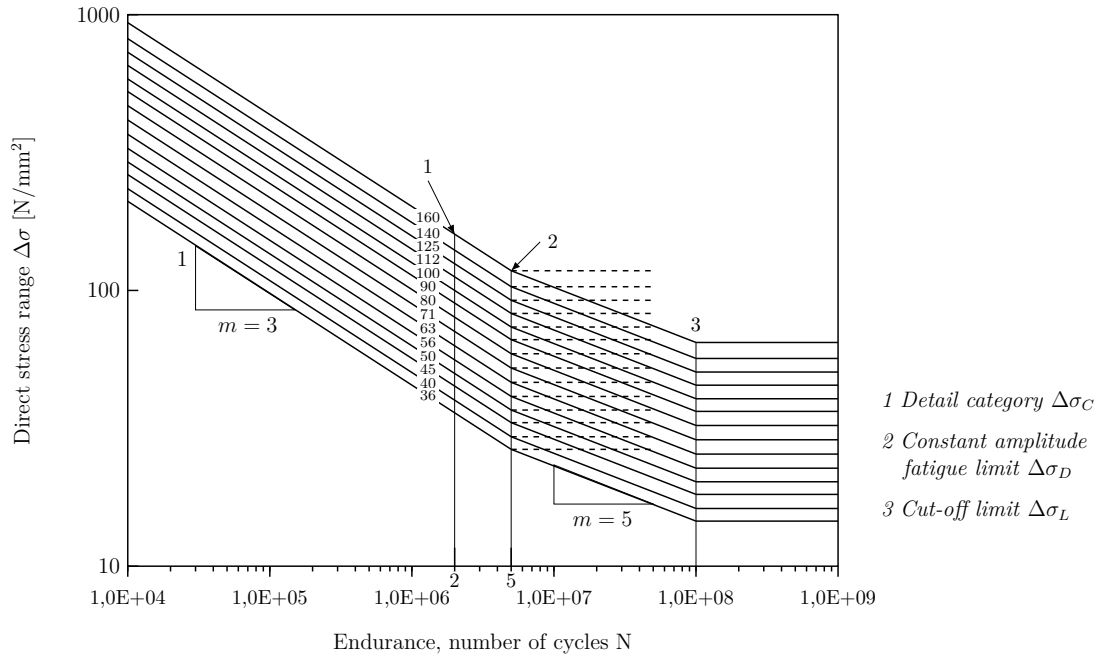


Figure 2.1: Characteristics of the S-N curve for AW details from *Eurocode 3: Design of steel structures- Part 1-9: Fatigue* (Swedish Standards Institute, 2005c).

Fatigue is usually the governing limit state in the design of railway bridges (Al-Emrani & Aygöl, 2014). To avoid that this causes heavy and non economical designs, fatigue design for railway bridges is to be done using "safe life design". Either the damage accumulative approach (such as equivalent stress range method) or the λ -coefficient method could be used for FLS verification. For the λ -coefficient method

LM71 (see Figure 2.2), SW/0 and SW2 could to be used with the concentrated load placed to obtain the worst case of internal force for each detail studied. When the damage accumulation method is used, "traffic mixes" is used instead of load models. The different combinations of traffic mixes could be found in *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003), alternatively national defined traffic mix defined in *TRVK Bro 11* (Trafikverket, 2011).

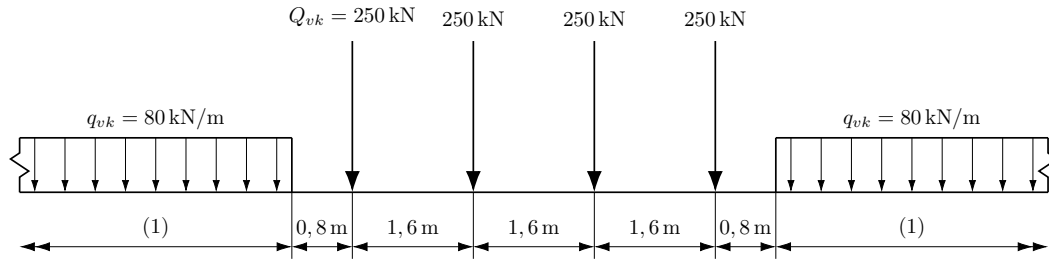


Figure 2.2: LM71 and characteristic values for vertical loads from *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003).

2.1.2 Post Weld Treatment

As mentioned earlier, FLS is often the limiting state in railway bridges. With fatigue being the governing limit state, this often leads to bulky designs, since the design requirements are often fulfilled by increased dimensions. Alternatives to purely increasing plate dimensions, to enhance fatigue capacity, is beneficial to improve fatigue strength (Haagensen & Maddox, 2004). There are several different types of PWT-methods that could be used to achieve an improved fatigue strength. For example, Burr Grinding (BG), Tungsten Inert Gas (TIG) dressing, and different peening methods, with HFMI being one of the newer versions of peening (Shams-Hakimi et al., 2015). All of the mentioned methods work at least by smoothening the transition from weld to base material and removal of weld defects (see Figure 2.3), with some additional effects introduced for some PWT-methods, such as HFMI. The basic effects of PWT reduces stress concentrations and extends the crack initiation phase, which increases the fatigue life. Although all PWT-methods increase fatigue strength, BG and TIG-dressing have some downsides of possibly being more time- and energy consuming (Al-Karawi et al., 2023).



Figure 2.3: Weld toe in AW, BG and TIG dressed state, from Shams-Hakimi et al., 2015.

2.2 High Frequency Mechanical Impact

As mentioned above, HFMI is a PWT technique for welds that is intended to increase the fatigue strength of the welded components. The method is considered a further development of conventional hammer peening but is considered to be more easy-to-use and effective (Marquis & Barsoum, 2016).

The technology involves accelerating cylindrical indenters made of hardened HSS against the weld transition at a high frequency, usually around 90 Hz (Marquis & Barsoum, 2016). This reduces tensile residual stresses at the weld toe by creating a smooth indentation along the weld, as shown in Figure 2.4. This offset improves the local geometry of the weld by increasing the radius of the weld tip, reducing the stress concentration factor (Shams-Hakimi et al., 2015). It should be mentioned that HFMI is only an improvement in the fatigue capacity of the weld toe (Marquis & Barsoum, 2016), and can only be applicable to plates with a thickness greater than 5 mm according to *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)* (Swedish Standards Institute, 2022). Improving the weld toe has proven to be the most efficient way to improve fatigue capacity in most cases. However, it is worth mentioning that other crack initiation spots, such as the weld root, could be governing and that this cannot be improved by HFMI. Since increasing the capacity of the weld toe when designing with HFMI, it is important to remember to control other details that could become limiting.

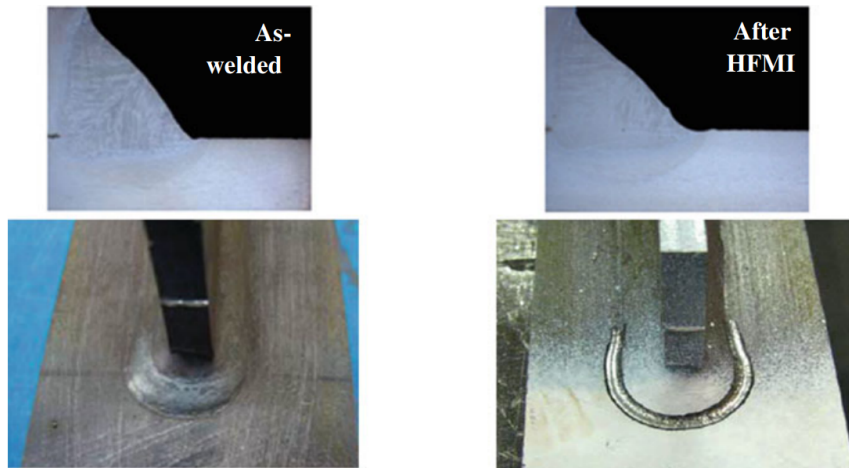


Figure 2.4: Weld toe before and after HFMI treatment (Marquis & Barsoum, 2016).

In addition to geometric improvement, the treatment reduces tensile residual stresses and introduces compressive residual stresses at the toe region of the weld (Al-Emrani et al., 2024). These compressive stresses replace the tensile stresses that occur during the welding process. They are considered to be the most important factor in increasing fatigue strength using HFMI treatment (Al-Karawi, 2023b). In addition to the favourable residual stresses, local plastic deformation leads to hardening of the material, increasing its resistance to cracking (Marquis & Barsoum, 2016).

2.2.1 Benefits with Implementing HFMI

The implementation of HFMI offers significant advantages in the design and maintenance of steel structures, especially for railway bridges, where fatigue is often the limiting factor. The primary benefit is the significant increase in the fatigue strength of the welded components. According to current recommendations, HFMI treatment can significantly increase the fatigue class (FAT) compared to untreated welds (Shams-Hakimi et al., 2015). This increase is related to the steel's yield strength, which means that a higher strength in the base material results in a greater increase in fatigue capacity after treatment. According to Marquis and Barsoum, 2016, it is estimated that for every 200 MPa increase in static yield strength, the fatigue strength increases by one FAT class (approximately 12.5% increase in fatigue capacity, $\Delta\sigma$).

A direct consequence of increased fatigue strength is the possibility of weight optimization. By utilizing the higher capacity, the dimensions of the load-bearing elements can be reduced, leading to lower material consumption and lower self weight. Case studies on railway and road bridges have shown that material savings between 20% and 30% are possible, depending on the span and design (Shams-Hakimi et al., 2015). However, this study only consider the increase in the fatigue class of HFMI and the effect of higher grade steel, it does not include the effects of R-ratio and self weight (see Section 2.2.1.2). Therefore, even if the results are promising, more research is needed to confirm the results, including all the effects needed, according to the current design guidelines for HFMI. In addition, steel class S355 and S460, spans from 10-30 m, has been considered earlier. This leaves some room for further investigation to determine whether higher-grade steel allows for structural improvements in both similar and longer spans.

2.2.1.1 Modification of S-N Characteristics

The effects of HFMI could be beneficial for fatigue design for stress ranges lower than $\Delta\sigma_S$ ($\Delta\sigma \leq \Delta\sigma_S$) and more load cycles than $N_{min,HFMI}$ ($N \geq N_{min,HFMI}$). Otherwise, the S-N curve for AW detail is to be used. $\Delta\sigma_S$ is determined from the fatigue strength for each detail category after HFMI treatment (obtained from the reference value $\Delta\sigma_{C,HFMI,ref}$ from *Eurocode 3 Design of steel structures Part 1-9: Fatigue (second generation)* (Swedish Standards Institute, 2022), which is to be adjusted according to Equation 2.5), $\Delta\sigma_C$ and Equation 2.1.

$$\Delta\sigma_s = \left(\frac{\Delta\sigma_{C,HFMI}^5}{\Delta\sigma_c^3} \right)^{0.5} \quad (2.1)$$

The S-N curve, as defined earlier, is the stress range plotted against the number of stress cycles on a log-log scale. For stress ranges greater than $\Delta\sigma_S$ there is a slope of 1:5 until the knee point at 5 million cycles (stress range $\Delta\sigma_{D,HFMI}$ defined by

Equation 2.2) whereupon a slope of 1:9 continuous to the cut-off limit at 100 million cycles (stress range $\Delta\sigma_{L,HFMI}$ defined by Equation 2.3).

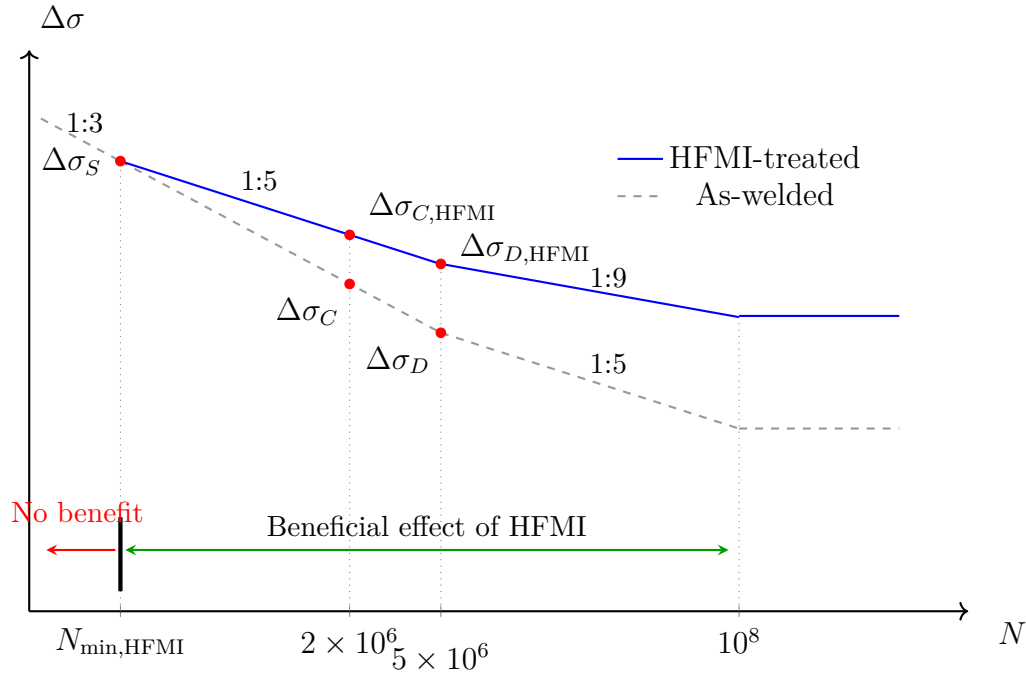


Figure 2.5: Changes in characteristics of the S-N curve from AW to HFMI-treated conditions (Al-Karawi et al., 2023).

While Figure 2.5 illustrates the improvement in fatigue behaviour used in the new *Eurocode 3 Design of steel structures Part 1-9: Fatigue (second generation)* (Swedish Standards Institute, 2022), the specific design parameters differ between the other available guidelines (Al-Karawi, 2023b). Figure 2.6 presents a comparison between the International Institute of Welding (IIW) recommendations and the German Committee for Steel Construction (DAST) guidelines (which are in accordance with *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)* (Swedish Standards Institute, 2022)). As shown in Figure 2.6, the IIW recommendation employs a S-N curve with other characteristics. In this fatigue resistance model, the knee point is situated at $N = 10^7$ cycles and unlike other standards, it does not define a discontinuation limit. This means that all stress cycles contribute to damage accumulation. The DAST guideline on the other hand, has its knee point at $N = 5 \cdot 10^6$ cycles and introduces a variable amplitude fatigue cut-off limit at $N = 10^8$ cycles, below which stress intervals are assumed not to cause fatigue damage. The used S-N curve throughout this thesis is the one recommended by DAST, since it is the one Eurocode recommends. The equations in all of Chapter 2 are according to the DAST guidelines.

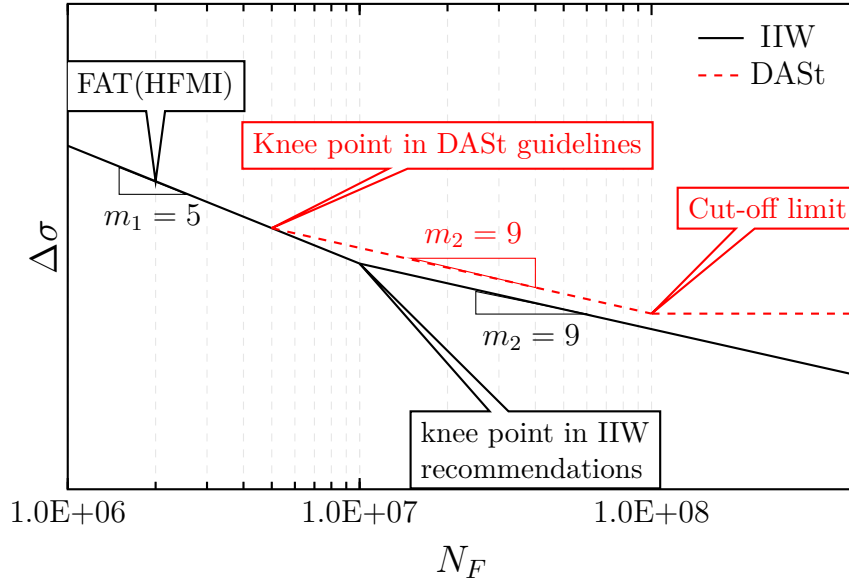


Figure 2.6: Design S-N curves for HFMI treatment according to IIW and DAST (Al-Karawi, 2023b).

The effects of shallower slopes of the S-N curves practically allow for a higher stress range at a certain number of stress cycles or a larger number of cycles at a certain stress range.

$$\Delta\sigma_{D,HFMI} = \left(\frac{2}{5}\right)^{\frac{1}{5}} \Delta\sigma_{C,HFMI} = 0.833 \Delta\sigma_{C,HFMI} \quad (2.2)$$

$$\Delta\sigma_{L,HFMI} = \left(\frac{5}{100}\right)^{\frac{1}{9}} \Delta\sigma_{D,HFMI} = 0.717 \Delta\sigma_{D,HFMI} \quad (2.3)$$

$$N_{\min,HFMI} = 2 \cdot 10^6 \left(\frac{\Delta\sigma_c}{\Delta\sigma_s}\right)^3 = 2 \cdot 10^6 \left(\frac{\Delta\sigma_{C,HFMI}}{\Delta\sigma_s}\right)^5 \quad (2.4)$$

Since HFMI treatment only enhances fatigue capacity locally at the weld toe, its effectiveness is inherently limited to details where failure originates from this specific location. Consequently, the application of HFMI is currently restricted to specific structural details, such as transverse stiffeners, transverse butt welds, and longitudinal stiffeners (see Figure 2.7), according to *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)* (Swedish Standards Institute, 2022).

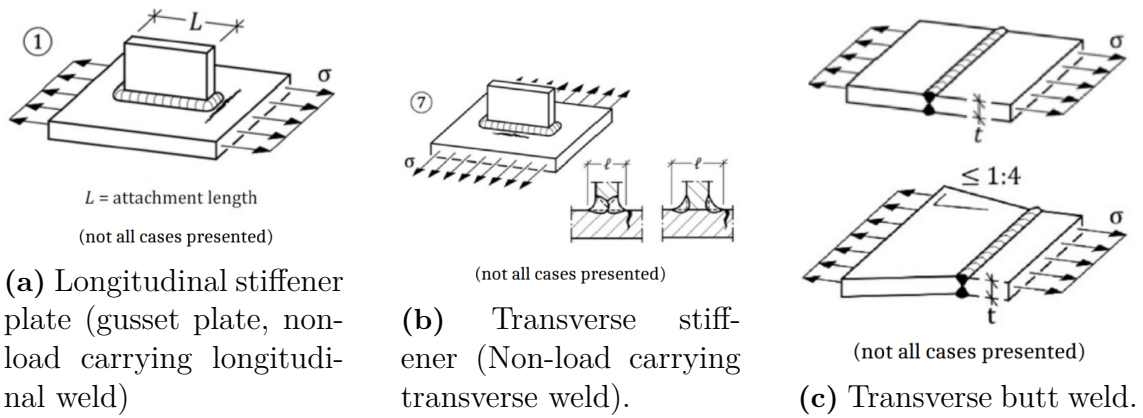


Figure 2.7: Details that could benefit from implementation HFMI according to the new *Eurocode 3 — Design of steel structures — Part 1-9: Fatigue (second generation)* (Swedish Standards Institute, 2022).

2.2.1.2 Impact of Steel Class on HFMI

One of the characteristic features of the HFMI treatment is that the improvement in fatigue strength depends on the yield strength of the steel material (Al-Emrani et al., 2024). This is different from untreated welds, where fatigue strength is generally considered to be independent of steel class. HFMI-treated parts, on the other hand, show higher fatigue resistance when higher strength steel is used (Al-Karawi, 2023b). According to IIW, an HFMI-treated part can be improved with up to 8 FAT classes, see Figure 2.8. The degree of this improvement depends on factors such as the strength of the material and the geometry of the weld (Marquis & Barsoum, 2016). It also depends on the R-ratio described in Section 2.2.2.3.

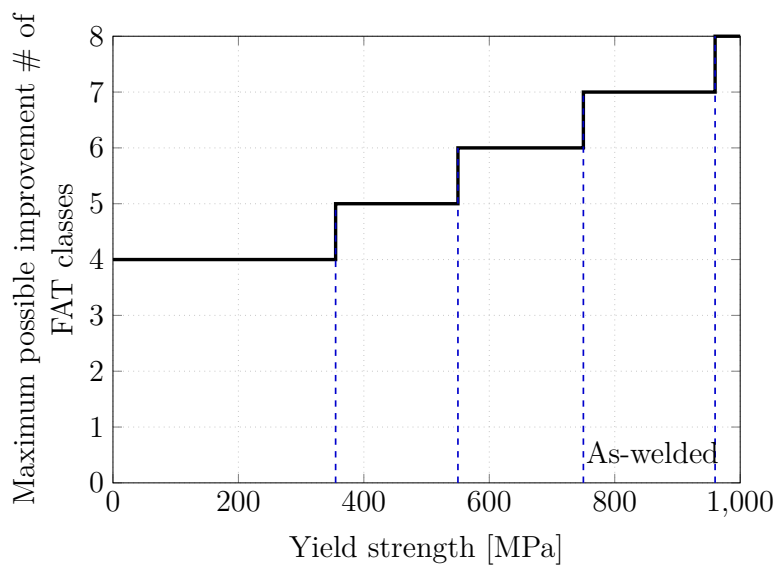


Figure 2.8: Maximum increase in number of FAT classes as a function of f_y (Marquis & Barsoum, 2016).

As mentioned in Section 2.2.1, the fatigue strength improvement is directly related to the yield strength of the base material. To take this into account in the design, a modification factor, f_1 , is introduced, which modifies the reference value of the fatigue strength ($\Delta\sigma_{C,HFMI,ref}$) based on the actual yield strength (f_y). This correlation is described by Al-Emrani et al., 2024 as follows:

$$\Delta\sigma_{C,HFMI} = f_1 f_2 \Delta\sigma_{C,HFMI,ref} \quad (2.5)$$

$$f_1 = 1 + \frac{0.1(f_y - 355)}{\Delta\sigma_{C,HFMI,ref}} \quad (2.6)$$

In Equation 2.6, f_y is the yield strength of the steel and $\Delta\sigma_{C,HFMI,ref}$ is the reference value for the fatigue strength of a certain detail with steel class S355. This correlation allows for the use of High-Strength Steel (HSS) more efficiently in bridge construction, which could potentially lead to lighter structures and material savings.

2.2.2 Challenges with HFMI

In untreated welds, fatigue strength is often assumed to be independent of load conditions. This is due to the high residual tensile stresses already present at the weld toe. HFMI-treated components are sensitive to load history (Al-Karawi, 2023a). As a consequence of this, a penalty factor is applied, as described in Section 2.2.1.2, for constant amplitude loading. The main challenge in design is to ensure the stability of the favourable compressive residual stresses induced by the HFMI treatment.

If the favourable compressive residual stresses relax (decrease or disappear), for example due to overload effects, during the intended lifespan of the structure, the expected service life extension is lost. The two main factors that can lead to this relaxation are high stress ratios (high mean stress) and the occurrence of high peak stresses (overload effects), especially compressive peak stresses (Al-Karawi, 2023a).

2.2.2.1 Impact of Plate Thickness on HFMI

Just like AW details, HFMI-treated welds are subject to the so-called ‘size effect’, which means that the fatigue strength decreases as the thickness of the plate increases (Shams-Hakimi et al., 2017). This phenomenon occurs because thicker plates have a lower stress gradient at the weld toe compared to thinner plates. Therefore, a larger volume of material is exposed to high stresses, which statistically increases the probability of a defect or weak point where a crack can occur.

To account for this when dimensioning HFMI-treated details, a reduction factor, k_s , is used. According to Annex F in the new revision of *Eurocode 3 Design of steel structures Part 1-9: Fatigue (second generation)* (Swedish Standards Institute, 2022) and in the bridge design recommendations of Al-Emrani et al., 2024, the fatigue strength should be reduced if the thickness of the plate (t) exceeds 25 mm. The reduction factor is calculated using the following formula:

$$k_s = \left(\frac{25}{t}\right)^{0.2} \quad \text{for } t > 25 \text{ mm} \quad (2.7)$$

In practical terms, this exponent represents a moderate but significant reduction in the capacity of thick plates. For example, if the plate thickness doubled from 25 mm to 50 mm, the factor k_s would be approximately 0.87. This means that only 87% of the reference fatigue strength can be used for the thicker plate, ensuring that the structure remains safe despite the increased risk associated with larger material volumes.

2.2.2.2 Impact of Overload Effects on HFMI

A critical aspect for HFMI-treated components is their sensitivity to overload effects. Although treatment provides significant benefits, even a few cycles of high-magnitude loading have been shown to cause relaxation of the beneficial residual stresses (due to overload effects) at the weld toe. Compressive overloads are found to be harmful to positive residual stresses (Shams-Hakimi et al., 2015), as they can more effectively lead to local yielding and subsequent stress dissipation (Al-Emrani et al., 2024). This phenomenon occurs because once an external compressive load is added to the existing compressive residual stresses from the HFMI treatment, the material can reach its yield strength locally (local yielding). This leads to local plastic deformations, resulting in residual stresses being reduced or completely eliminated. Specific limit values have been defined to ensure that the HFMI treatment retains its effectiveness throughout the service life of the structure (Marquis & Barsoum, 2016). According to Al-Emrani et al., 2024, the maximum stress ($\sigma_{\max}$) of the characteristic load combination in SLS should remain within the limits shown in Table 2.1, depending on the type of welded detail:

Table 2.1: Recommended maximum nominal compressive stress (deified as negative values) limits for different weld details (Al-Emrani et al., 2024).

Weld Detail	Stress Limit Condition
Butt welds	$-0.9f_y \leq \sigma_{\max} \leq f_y$
Transverse stiffener	$-0.7f_y \leq \sigma_{\max} \leq f_y$
Longitudinal stiffener	$-0.5f_y \leq \sigma_{\max} \leq f_y$

In addition to these stress limit conditions, the IIW also recommends that the maximum nominal compressive stress should not exceed $-0.8 \cdot f_y$. Al-Karawi, 2023a also mentions that the limitation is introduced to prevent local yielding, when superimposing residual compressive stresses with compressive stresses from loads. If not considered it could lead to relaxation at the weld toe, eliminating the favourable residual compressive stresses induced by HFMI treatment. If any of these limits are exceeded, the positive effect of HFMI treatment cannot be taken into account in design.

2.2.2.3 Stress Ratio and FAT Class Reduction

As mentioned above, the positive effects of HFMI are highly dependent on induced compressive residual stresses. It has been shown that a higher mean stress during

load cycles leads to less improvement of fatigue strength with HFMI treatment (Al-Emrani et al., 2024). The ratio between min and max stresses of the loading cycle also affects gains from HFMI. Methods accounting for this is calibrated according to some broadly used load models and dependent on whether HFMI treatment is done post- or pre bridge erection. The higher the R-ratio gets the more it penalizes the positive effects of HFMI more (Marquis & Barsoum, 2016). For constant amplitude stress ranges, the FAT class is reduced according to Table 2.2. In the calculations, this table is represented by the correction factor f_2 , as defined by Equation 2.22 for cases of variable amplitude loading (Al-Karawi et al., 2023). The Table 2.2 only considers R-ratio < 0.52 while the factor f_2 also consider higher R-ratios ($0.52 \leq R < 1$). For loads covered by *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003) (train mixes and LM71) the correction factor λ_{HFMI} (according to Equation 2.17 or 2.18) could be used to account for the R-ratio and the mean stress reduction of the FAT class.

Table 2.2: Stress ratio effect on fatigue strength class (Marquis & Barsoum, 2016).

Stress ratio	FAT class reduction
$R \leq 0.15$	0
$0.15 < R \leq 0.28$	1
$0.28 < R \leq 0.40$	2
$0.40 < R \leq 0.52$	3
$0.52 < R$	No available data

$$\frac{\Delta\sigma_{Ed}}{\Delta\sigma_{C,HFMI}/\gamma_{Mf}} \leq 1.0, \text{ \& } \Delta\sigma_{Ed} < \frac{\Delta\sigma_s}{\gamma_{Mf}} \quad (2.8)$$

$$\Delta\sigma_{Ed} = \Delta\sigma_E \gamma_{Ff} \quad (2.9)$$

In the case of a non-variable stress range for HFMI-treated details, Equation 2.8 can be utilized to assess the fatigue resistance of specific details (Al-Emrani et al., 2024). Railway bridges subject to variable amplitude loading require consideration of the variation, as well as self weight if HFMI treatment is conducted in workshop before bridge erection.

2.2.2.4 Fatigue Design for Variable Amplitude Loading Using λ -Coefficient Method

As mentioned earlier, there are a couple of ways to consider the variable amplitude loading of the welded details and the same applies for HFMI-treated details. Some different methods are appropriate depending on the used load model and the choice of design approach (Al-Emrani et al., 2024). When LM71 is used for fatigue design λ -coefficient method is appropriate to use, in an initial design, when it is not fully determined how the load spectrum will end up. The λ -coefficient method is adjusted to reflect the R-ratio and mean stress effects, that arise from HFMI treatment,

through the introduction of a specific correction factor, λ_{HFMI} (Al-Karawi et al., 2023).

$$\frac{\Delta\sigma_{e,2,HFMI,Ed}}{f_1 \times \Delta\sigma_{C,HFMI,ref}/\gamma_{Mf}} < 1 \quad (2.10)$$

$$\Delta\sigma_{e,2,HFMI,Ed} = \lambda_1\lambda_2\lambda_3\lambda_4\lambda_{HFMI}\Delta\sigma_{Ed} \quad \& \quad \lambda_1\lambda_2\lambda_3\lambda_4 < \lambda_{Max} \quad (2.11)$$

The magnification factor, λ_{HFMI} , consider both the effects of the R-ratio and the self weight of the structure. The self weight is considered through Φ from Equation 2.19, which is then used to calculate λ_{HFMI} . λ_{HFMI} is calculated according to Equation 2.17 or 2.18 for mid span and mid support respectively. Equations 2.17 and 2.18 are derived and calibrated against measured traffic data (Al-Emrani et al., 2024).

2.2.2.5 Fatigue Design for Variable Amplitude Loading Using Damage Accumulation Method

Another method to consider variable amplitude loading is the damage accumulation method, as mentioned in Section 2.1.1. The nature of the damage accumulation method is to verify fatigue resistance by calculating the total accumulated fatigue damage. The damage accumulation method is used when the traffic categories and load patterns acting on a bridge are known or estimated through standardized models. For instance, models as train mixes according to *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003). The verification is a summation of the damage to ensure it does not exceed the allowed limit, as defined by Equation 2.12.

$$D = \frac{\Sigma n}{N_{eq}} \leq 1.0 \quad (2.12)$$

The equivalent number of stress cycles N_{eq} is calculated according to Equation 2.14, if there are stress ranges on both sides of the knee point. Equation 2.16 should be used if there are only stress ranges on one side of the knee point. Calculating the equivalent stress range needed for either Equation 2.13 or 2.15 is used for the respective case. In the case where traffic mixes according to *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* is used, R-ratio and self weight are considered through the correction-factor λ_{HFMI} similarly as with the λ -coefficient method. In contrast to when the λ -coefficient method is used, Φ is to be calculated according to Equation 2.20, when traffic mixes and damage accumulation method are used.

$$\Delta\sigma_{eq} = \begin{cases} \sqrt[m_1]{\frac{\Sigma(n_i\Delta\sigma_i^{m_1}) + \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}\right)^{m_1-m_2} \cdot \Sigma(n_j\Delta\sigma_j^{m_2})}{\Sigma n_i + \Sigma n_j}} & , \text{ if } \frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} \leq \Delta\sigma_{eq} \\ \sqrt[m_2]{\frac{\Sigma(n_i\Delta\sigma_i^{m_1}) \cdot \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}\right)^{m_2-m_1} + \Sigma(n_j\Delta\sigma_j^{m_2})}{\Sigma n_i + \Sigma n_j}} & , \text{ if } \frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} > \Delta\sigma_{eq} \end{cases} \quad (2.13)$$

$$N_{eq} = \begin{cases} 5 \cdot 10^6 \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}/\gamma_{Mf}}{\lambda_{HFMI} \cdot \Delta\sigma_{eq} \cdot \gamma_{Ff}} \right)^{m_1} & , \text{ if } \frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} \leq \lambda_{HFMI} \cdot \Delta\sigma_{eq} \\ 5 \cdot 10^6 \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}/\gamma_{Mf}}{\lambda_{HFMI} \cdot \Delta\sigma_{eq} \cdot \gamma_{Ff}} \right)^{m_2} & , \text{ if } \frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} > \lambda_{HFMI} \cdot \Delta\sigma_{eq} \end{cases} \quad (2.14)$$

In Equations 2.13 and 2.14 (Al-Emrani et al., 2024)

m_1, m_2 are the slopes of the S-N curve, i.e. 5 and 9 respectively

$\Delta\sigma_i, \Delta\sigma_j$ are the stress range above and below the reference stress at the knee point.

knee point is defined as $f_1 \Delta\sigma_{D,HFMI,ref}/\gamma_{Mf}$.

In the case of stress ranges on one side of the knee point m in Equation 2.15 and 2.16 is either m_1 or m_2 .

$$\Delta\sigma_{eq} = \sqrt[m]{\frac{\Sigma(n \cdot \Delta\sigma_i^m)}{\Sigma n}} \quad (2.15)$$

$$N_{eq} = 5 \cdot 10^6 \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}/\gamma_{Mf}}{\lambda_{HFMI} \cdot \Delta\sigma_{eq} \cdot \gamma_{Ff}} \right)^m \quad (2.16)$$

$$\lambda_{HFMI} = \frac{2.38\Phi + 1.18}{\Phi + 1.07} \geq 1.0 \quad \text{Midspan} \quad (2.17)$$

$$\lambda_{HFMI} = \frac{2.56\Phi + 1.12}{\Phi + 1.61} \geq 1.0 \quad \text{Mid-support} \quad (2.18)$$

$$\Phi = \frac{\sigma_{perm}}{0.73 \times \Delta\sigma_{LM71}} \quad (2.19)$$

$$\Phi = \frac{\sigma_{perm}}{0.90 \times \Delta\sigma_{max}} \quad (2.20)$$

In Equations 2.19 and 2.20 (Al-Emrani et al., 2024)

$\Delta\sigma_{LM71}$ are the max stress range generated from LM71,

$\Delta\sigma_{max}$ are the max stress range generated from the passage of different trains included in train mixes from *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003).

The damage accumulation method could also be used for other loads than those covered by *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003). However, that requires some different adjustments when combined with HFMI treatment. As mentioned above, λ_{HFMI} could only be used in combination with loads covered by *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges*, since it is based on specific measured traffic data covered in *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Al-Emrani et al., 2024). In other cases, the effects of the R-ratio and self weight could be considered directly through R from Equation 2.21 and the factor f_2 according to Equation 2.22. In this case where the R-ratio is calculated according to Equation 2.21, σ_{perm} is the stress from the self weight of the bridge, σ_{min} the minimum stress from non-permanent load and σ_{max} the maximum stress from non-permanent load. The R-ratio is calculated individually for each detail and the corresponding section

where it is placed. When HFMI is applied on site, σ_{perm} should be set to zero (Al-Karawi et al., 2023), which suggests that the effect of the R-ratio could be ignored if HFMI treatment is conducted after the assembly of the bridge.

$$R_i = \frac{\sigma_{\min}}{\sigma_{\max}} = \frac{\sigma_{\min.i} + \sigma_{\text{perm}}}{\sigma_{\max.i} + \sigma_{\text{perm}}} \quad (2.21)$$

$$f_2 = \frac{1}{0.5R^2 + 0.95R + 0.9} \quad \text{if } 0.1 < R < 1.0, \text{ otherwise, } f_2 = 1.0 \quad (2.22)$$

Verification of total damage with loads not covered by *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* is still done according to Equation 2.12, but $\Delta\sigma_{eq,R}$ and N_{eq} are to be calculated according to Equations 2.23 and 2.24 including R-ratio effects through the correction factor f_2 .

$$\Delta\sigma_{eq,R} = \begin{cases} \sqrt[m_1]{\frac{\Sigma\left(n_i \left[\frac{\Delta\sigma_i}{f_{2,i}}\right]^{m_1}\right) + \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}\right)^{m_1-m_2} \cdot \Sigma\left(n_j \left[\frac{\Delta\sigma_j}{f_{2,j}}\right]^{m_2}\right)}{\Sigma n_i + \Sigma n_j}} & \text{if } \frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} \leq \Delta\sigma_{eq,R} \\ \sqrt[m_2]{\frac{\Sigma\left(n_i \left[\frac{\Delta\sigma_i}{f_{2,i}}\right]^{m_1}\right) \cdot \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}\right)^{m_2-m_1} + \Sigma\left(n_j \left[\frac{\Delta\sigma_j}{f_{2,j}}\right]^{m_2}\right)}{\Sigma n_i + \Sigma n_j}} & \text{if } \frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} > \Delta\sigma_{eq,R} \end{cases} \quad (2.23)$$

$$N_{eq} = \begin{cases} 5 \cdot 10^6 \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref} / \gamma_{Mf}}{\Delta\sigma_{eq,R} \cdot \gamma_{Ff}} \right)^{m_1} & \text{if } \frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} \leq \Delta\sigma_{eq,R} \\ 5 \cdot 10^6 \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref} / \gamma_{Mf}}{\Delta\sigma_{eq,R} \cdot \gamma_{Ff}} \right)^{m_2} & \text{if } \frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} > \Delta\sigma_{eq,R} \end{cases} \quad (2.24)$$

3

Methods

The methodology is divided into two main categories: hand calculations and the PDM developed in MATLAB. Since the thesis aims to optimize bridge cross-sections both with and without the implementation of HFMI treatment, this needs to be conducted using a computational optimization tool, which in this case is a GA. To verify the functions within the PDM, hand calculations have been performed, which subsequently were verified against the original design calculations for the bridge conducted by COWI. Hand calculations have, to some extent, been simplified compared to the original bridge calculations, as the original design was partly performed using a finite element model (FEM).

3.1 Structural System Model

To accurately capture the structural response of the bridge under various load effects, the mechanical behaviour of the bridge is modelled using a combination of global 1D and 2D finite element idealizations, supplemented by localized analytical mechanical models. The procedure used to cover all necessary effects are presented by the flowchart in Figure 3.1.

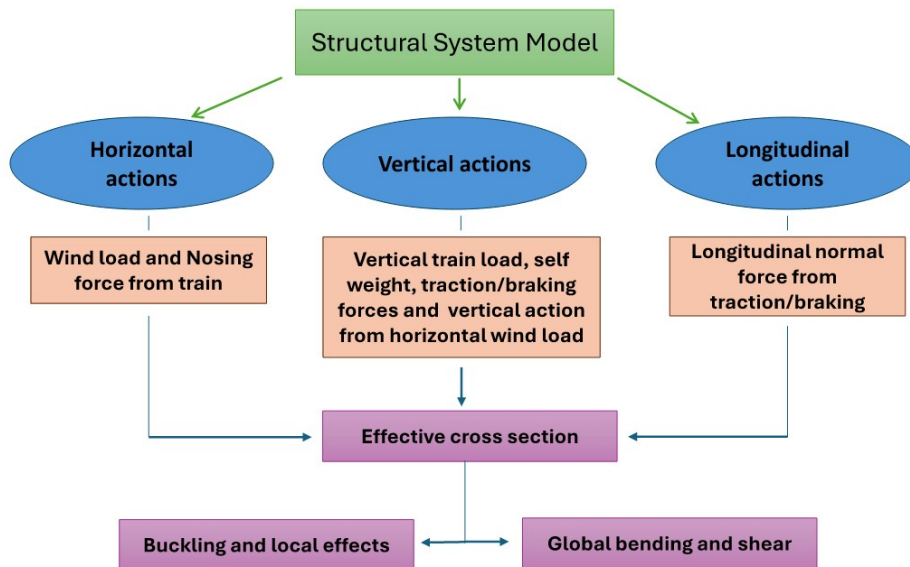


Figure 3.1: Flowchart of Structural System Model.

Global Vertical and Longitudinal Behaviour

The main girders are idealized as Euler-Bernoulli beams with variable flexural stiffness. They primarily resist vertical bending moments and shear forces induced by self weight, traffic loads (see Figures 3.2 and 3.3), and wind loads (see Figure 3.4). Regarding boundary conditions, the bridge is assumed to be simply supported, where one support acts as longitudinally fixed, while the others allow longitudinal translation. Consequently, longitudinal actions, such as traction and braking forces, accumulate towards the fixed support (see Figure 3.3). The structural model is formulated to be fully reversible, accounting for bi-directional traffic and allowing the fixed bearing to be located at either end of the bridge. Due to the eccentricity of these loads relative to the section's centre of gravity, they induce a combination of axial forces and secondary vertical bending moments in the main girders (with the eccentricity e_{normal} shown in Figure 3.3). Furthermore, transverse wind loads acting on the bridge and the traffic are translated into an overturning moment about the bridge's centre of gravity (see Figure 3.4), which is resisted by the main girders as a vertical force couple.

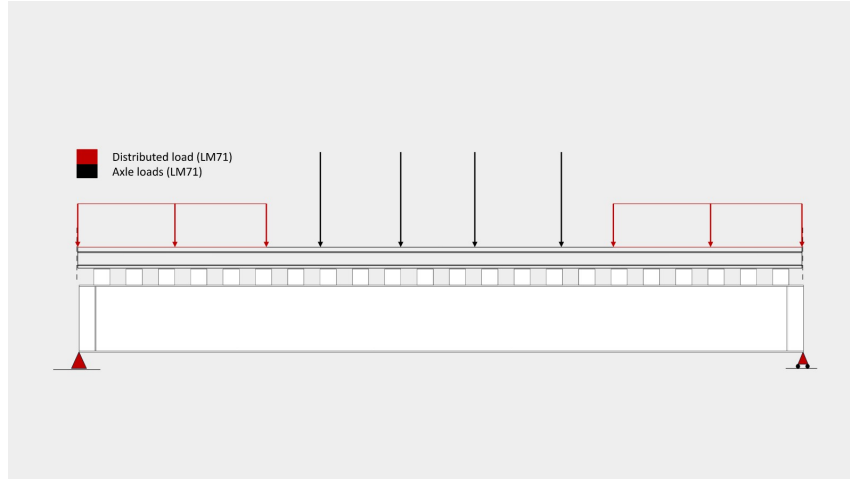


Figure 3.2: Application of vertical train load.

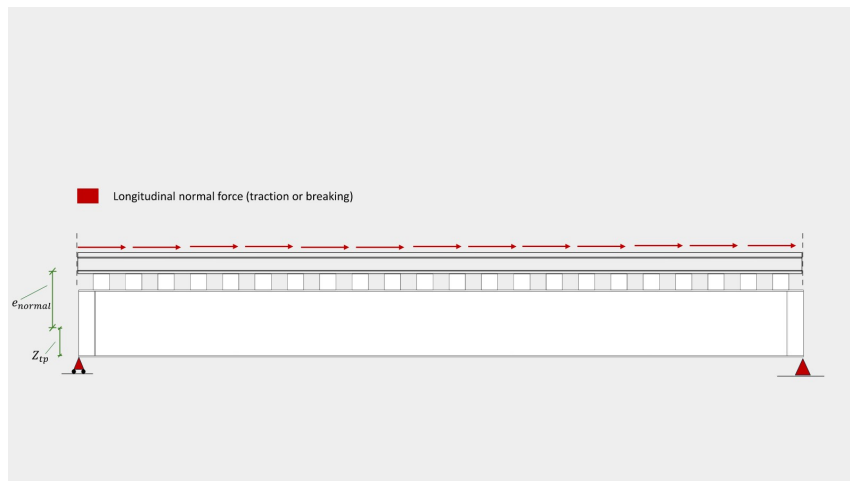


Figure 3.3: Application of longitudinal train load (traction and braking load).

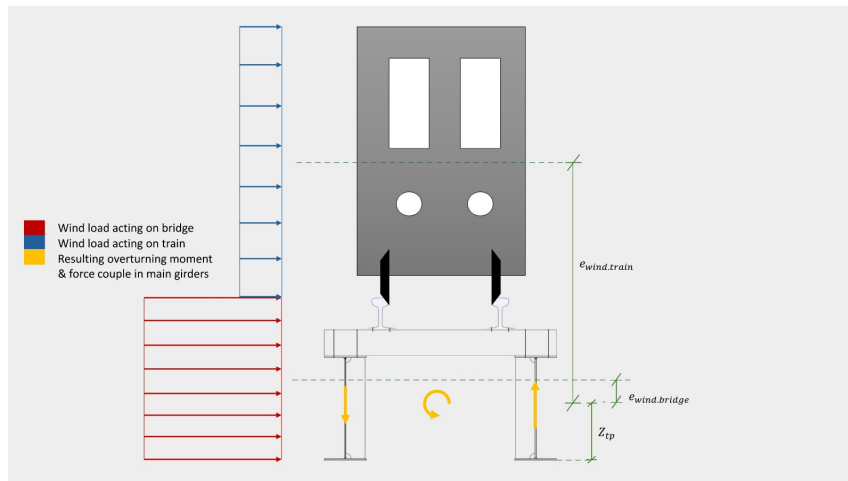


Figure 3.4: Application of horizontal wind load acting on train and main beams generating a overturning moment absorbed as a force couple.

Transverse Behaviour and Bracing System

Transverse horizontal actions, namely wind pressure and localized train nosing forces (see Figures 3.5 and 3.6), resist the horizontal bracing system together with the main girders. This system is evaluated using a 2D frame model. In this model, the UPE cross-beams and L-profile diagonals are idealized as pinned truss elements, meaning they possess no flexural stiffness and only transfer axial forces. The main girders, which act as the chords of the frame, are modelled with lateral flexural stiffness. Nosing forces induce local lateral bending in the main girders, which is evaluated by assuming the individual girder acts as a simply supported beam spanning between the rigid transverse cross-girders.

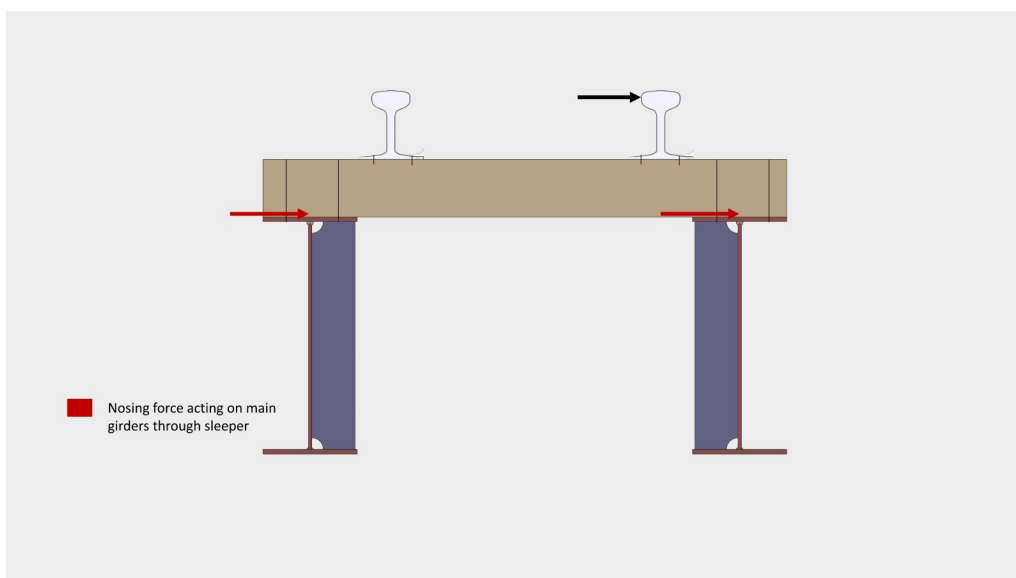


Figure 3.5: Application of horizontal nosing force from train.

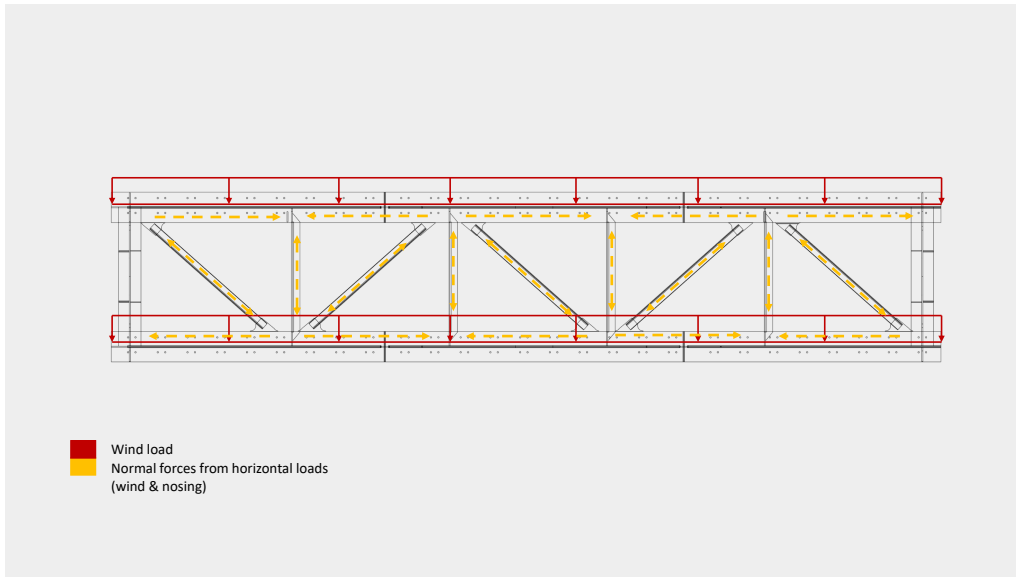


Figure 3.6: Application of horizontal loads absorbed as normal forces by the main beams through the truss system.

Cross-Sectional Mechanics

At the cross-sectional level, global bending moments are primarily resisted by the flanges, while the web plates resist vertical shear flow and vertical concentrated patch loads from the train axles. The axial forces originate from both longitudinal actions and horizontal loads transferred to the main girders by the bracing system. These forces are assumed to be uniformly distributed over the full cross-section as normal stresses. Transverse stiffeners are modelled to act as rigid boundaries to restrict shear buckling of the web panels. In the post-buckling regime, they are structurally evaluated as effective columns carrying compressive forces induced by tension field action. Finally, individual truss members are evaluated for gross section yielding under tension and for global flexural buckling under compression.

3.2 Hand Calculation

To verify the load functions implemented in MATLAB for the optimization process, hand calculations were used together with pre-verified tools such as PointSketch. PointSketch is a software for simple structural analysis developed at Chalmers University of Technology. It features a graphical user interface designed for the simulation and evaluation of truss based structures (Chalmers tekniska högskola, n.d.).

Preformed hand calculations (see Appendix A) are based on the relevant Eurocodes, *TRVINFRA-00227* and the design documents, based on Finite Element Analyses (FEA). The hand calculations and the relevant tools have been used as a preliminary verification tool to ensure the reliability of the MATLAB code. In Section 4.1, verification tables compare the hand calculations with the initial calculations, and the PDM model is then compared with the hand calculations. The initial design

geometry and load combinations (according to Table 3.1) were taken directly from the initial calculations to allow accurate comparisons. The limit state verification that has been considered is shown in Figure 3.7.

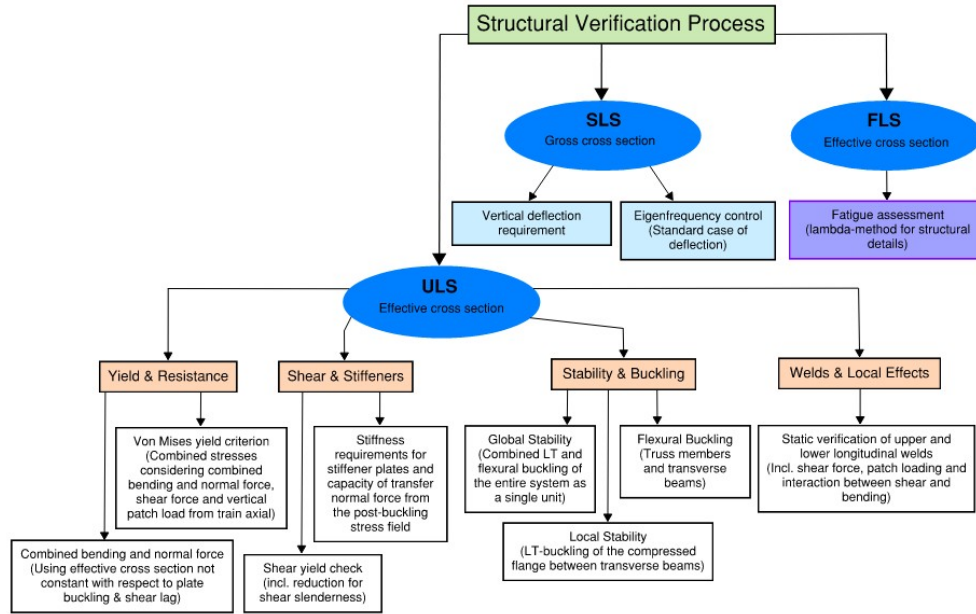


Figure 3.7: Flowchart for the limit state structural verifications.

More detailed, step-by-step explanations of the hand calculations can be found in Appendix A. Each load category and load combination taken into account in this verification process is described below.

3.2.1 Loads

For clarity, the loads acting on the bridge are categorized into three main groups:

- **Permanent load,**
 - Self weight of main girders,
 - Self weight of "Installations", Timber sleepers and rails (see Table 3.2),
- **Train load,**
 - LM71 (amplified by dynamic factor Φ),
 - Braking or Traction load (worst case),
 - Horizontal point load (nosing),
- **Other variable loads,**
 - Wind load.

Subsequently, these loads are combined using specific safety factors shown in Table 3.1 depending on the evaluated limit state, which is further detailed in Section 3.2.1.3. The dynamic factor Φ applied to LM71 is not applied to other loads such as real trains, fatigue load models, HSLM, or 'empty carriages' according to *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003).

Table 3.1: Load combinations according to *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003) and *Eurocode - Basis of structural design* (Swedish Standards Institute, 2002a).

Load combination factors	$\gamma_{G.self}$	$\gamma_{Q.LM71}$	$\gamma_{Q.traction}$	$\gamma_{Q.nosing}$	$\gamma_{Q.wind}$
ULS					
Traction as main	1.2	1.5	1.5	0.75	0.45
Nosing as main	1.2	1.5	0.75	1.5	0.45
SLS (Characteristic)					
Traction as main	1.0	1.0	1.0	0.5	0.3
Nosing as main	1.0	1.0	0.5	1.0	0.3
SLS (Frequent)					
Frequent (only train)	-	0.8	-	-	-

3.2.1.1 Permanent Loads

Permanent loads consist of the self weight of the structure and the weight of permanent installations. The self weight is calculated based on the density of the materials and the geometry of the structure, according to the *TRVINFRA-00227* and standard *Eurocode 1: Actions on structures Part 1-1: General actions - Densities, self weight, imposed loads for buildings* (Swedish Standards Institute, 2002b).

For structural steel, the specific weights given in Table A.4 from *Eurocode 1: Actions on structures Part 1-1: General actions - Densities, self weight, imposed loads for buildings* are used, while the values for wooden sleepers are taken directly from the initial calculations. The self weights for permanent track installations and safety features, including rails, gratings, and a standard railing, are based on Eurocode, specific product specifications, or assumed standard values. In addition, the weights for certain secondary steel sections (e.g. IPE, UPE, L-profile) have been taken from the manufacturers' structural tables Tibnor, n.d. All values for permanent loads used in the calculation are summarized in Table 3.2.

Table 3.2: Summary of self weights for materials and installations.

Item/Material	Unit Weight (γ)	Reference/Comment
<i>Materials</i>		
Steel	78.5 kN/m ³	<i>Eurocode 1: Actions on structures Part 1-1: General actions - Densities, self weight, imposed loads for buildings</i> Table A.4
Timber sleepers (assuming pine)	5.2 kN/m ³	Initial calculation

Continued from the previous page...

Table 3.2 – Continued from the previous page

Item/Material	Unit Weight (γ)	Reference/Comment
<i>Installations</i>		
Railing	1 kN/m	Assumed value
Rails (single track)	1.7 kN/m	<i>Eurocode 1: Actions on structures Part 1-1: General actions - Densities, self weight, imposed loads for buildings</i> , Table A.6
Grating	0.2 kN/m ²	Weight based on Weland H6x30, 1.3 m wide (Product H63021301001)
IPE 200	0.22 kN/m	From Tibnor, n.d. construction tables
UPE 400	0.709 kN/m	From Tibnor, n.d. construction tables
L100x75x12	0.152 kN/m	From Tibnor, n.d. construction tables
Vertical stiffener plates	0.215 kN/stiffener plate	Calculated

3.2.1.2 Variable Loads

Variable loads consist of loads varying both in magnitude and placement during the structural life span of the bridge. The main variable load consists of vertical and horizontal railway traffic. As mentioned in Section 2.1.1, the loads are based on LM71 according to *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* which represents normal rail traffic (Swedish Standards Institute, 2003). Figure 2.2 shows that the model consists of four point loads of 250 kN each and a distributed load of 80 kN/m.

To adjust LM71 to the specific conditions to which it is exposed, the characteristic forces are multiplied by a load magnification factor $\alpha_{traffic}$, and a dynamic factor Φ_2 . The dynamic factor takes into account the dynamic effects when the train passes and is calculated based on the determining length of the structural element (L_ϕ) and the maintenance of the track. If the bridge has a fixed single track, an eccentricity according to *TRVINFRA-00227* Table 7.1-7, "for fixed tracks, an eccentric track position with a lateral displacement of 0.05 m from the theoretical position must be considered" (Trafikverket, 2024) should be allowed for. The length of the structural element for the main girder is $L_\phi = 10.6$ m and for the cross girders $L_\phi = 3.0$ m. This gives different values for the dynamic factor, which is calculated in accordance with *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003). The values of the dynamic factors and the load magni-

fication factor are shown in Table 3.3.

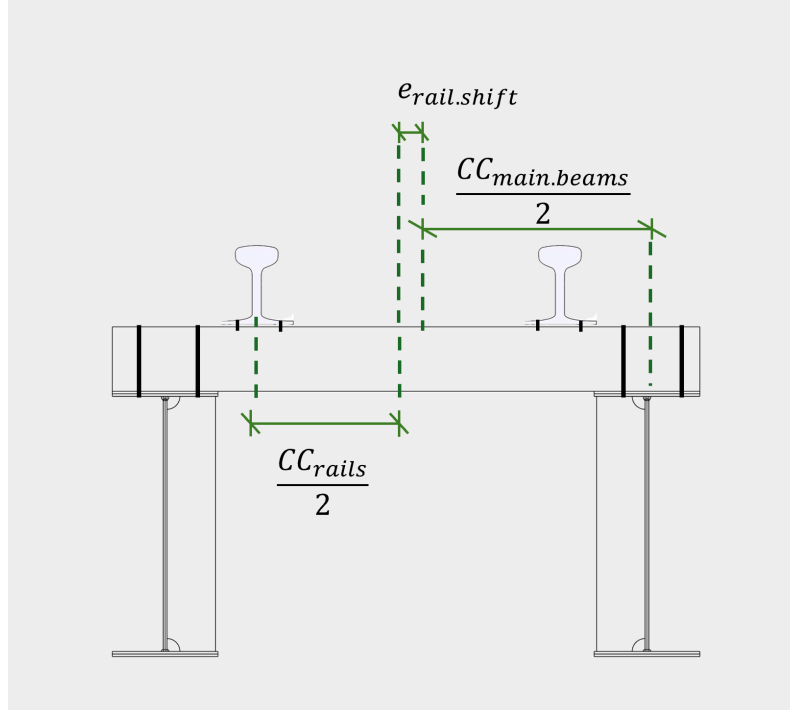


Figure 3.8: Rail shift eccentricity due to possible miss alignment of railway tracks.

$$\beta_{rs} = \frac{\frac{r}{2} + e_1}{r} \quad (3.1)$$

β_{rs} is the rail shift factor used to consider possible track eccentricity.
 r is the centre to centre distance between the rail tracks.
eccentricity for fixed track $e_1 = 0.05m$ (see $e_{rail.shift}$ Figure 3.8), according to *TRVINFRA-00227*.

When considering longitudinal forces, braking and acceleration forces the eccentricity from the centre of gravity to the top of the top flange of the main girders are taken into account. These forces are multiplied by $\alpha_{traffic}$ but not by the dynamic factor. The longitudinal design force $Q_{h,dim}$, is the highest value of the Equations 3.2 or 3.3. For stability checks, the ‘empty carriages’ load model with a distributed load of 10 kN/m is also taken into account. The displacement of eccentric load is taken into account in accordance with *TRVINFRA-00227* placing vertical loads with a displacement of $e = 0.05$ m from the track centre line.

Traction force:

$$Q_{lak} = 33 \text{ [kN/m]} \cdot L_{a,b} \text{ [m]} \leq 1000 \text{ [kN]} \quad (3.2)$$

For Load Models 71, SW/0, SW/2 and HSLM

Braking force:

$$Q_{lbk} = 20 \text{ [kN/m]} \cdot L_{a,b} \text{ [m]} \leq 6000 \text{ [kN]} \quad (3.3)$$

For Load Models 71, SW/0 and Load Model HSLM

Table 3.3: Summary of design parameters, forces, and geometry.

Parameter	Symbol	Value	Reference / Comment
<i>Factors</i>			
Rail shift factor	β_{rs}	0.54	According to <i>TRVINFRA-00227</i> and Equation 3.1
Load magnification factor	$\alpha_{traffic}$	1.33	TSFS 2018:57, ch. 11§ (Transportstyrelsen, 2018)
Dynamic factor (Main girder)	$\Phi_{2,MG}$	1.29	<i>Eurocode 1: Actions on structures - part 2: Traffic loads on bridges</i> (based on $L_\Phi = 10.6 \text{ m}$)
Dynamic factor (Cross girder)	$\Phi_{2,CG}$	1.67	<i>Eurocode 1: Actions on structures - part 2: Traffic loads on bridges</i> (based on $L_\Phi = 3.0 \text{ m}$)
<i>Forces</i>			
Vertical point load (LM71)	Q_{vk}	250 kN	Before factors applied
Vertical distributed load (LM71)	q_{vk}	80 kN/m	Before factors applied
Braking/Acceleration force	$Q_{h,dim}$	465 kN	Incl. $\alpha_{traffic}$ (Max value acc. to calc.)
Empty wagons (distributed load)	q_{empty}	10 kN/m	Used for stability check
<i>Geometry</i>			
Eccentricity	e	0.05 m	<i>TRVINFRA-00227</i>

Another variable load is the wind load. Wind load is calculated transversely according to *Eurocode 1: Actions on structures-part 1-4: General actions-wind actions* with additions from *TRVINFRA-00227*. The bridge is assumed to be located in terrain type II with a reference wind speed of 25 m/s. The dynamic response is not taken into account for this type of construction. The wind force is calculated mainly in the transverse direction (x-direction). The characteristic velocity pressure q_p , is taken from TSFS 2018:57 (Transportstyrelsen, 2018). The magnitude of the wind load also depends on the shape factor C_{fx} of the structure, which includes the influence of trains, and the reference area $A_{ref,x}$. C_{fx} and $A_{ref,x}$ are calculated according to *Eurocode 1: Actions on structures-part 1-4: General actions-wind actions* (Swedish Standards Institute, 2005a). Wind loads in the longitudinal direction of

the bridge have been neglected. This is because they are considered to be of minor importance in relation to the self-weight. The input data and the calculated wind loads are shown in Table 3.4.

Table 3.4: Summary of wind load parameters and results.

Parameter	Symbol	Value	Reference / Comment
<i>Input Data</i>			
Basic wind velocity	v_b	25 m/s	
Peak velocity pressure	q_p	0.64 kN/m ²	TSFS 2018:57 (Transportstyrelsen, 2018)
Force coefficient (incl. traffic)	c_{fx}	2.21	<i>Eurocode 1: Actions on structures-part 1-4: General actions-wind actions</i> , Fig 8.3
Reference area	$A_{ref,x}$	59.4 m ²	Based on 11 m length
<i>Results</i>			
Wind force (transverse)	$F_{w,x}$	84 kN	Characteristic value

3.2.1.3 Load Combination

Permanent and variable loads are combined into specific load cases in order to assess the structural integrity and functionality of the bridge. As previously shown in Table 3.1, different load combination factors (γ) are applied depending on the limit state and whether a load is considered a primary or secondary load. The calculations are divided into three primary limit states:

Ultimate Limit State

The ULS is verified to ensure the structural safety of the bridge and protect it against structural failure. In this limit state, the characteristic loads are multiplied by the safety factors specified in Table 3.1 to formulate appropriate load combinations according to *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003) and *Eurocode – Basis of structural design* (Swedish Standards Institute, 2002a). To determine the most critical load effect (the worst-case scenario), the maximum load factor is applied to one leading variable load at a time, while any accompanying variable loads are reduced accordingly. However, the train load is always considered a main load and the wind load is considered a secondary variable load.

Serviceability Limit State

The SLS deflection is evaluated to ensure that the bridge works properly under normal daily operating conditions. This includes checking for deformations that can affect travel comfort or track geometry. For these evaluations, the frequent load combination is utilized. Since the bridge is designed with precamber (an upward initial curvature), the vertical displacements caused by self weight are effectively counteracted and therefore not included in deflection calculation according to *Eurocode – Basis of structural design* (Swedish Standards Institute, 2002a). Furthermore, horizontal dynamic actions, such as wind loads, are excluded from these frequent SLS combinations. Since the simultaneous occurrence of extreme wind events and dominant vertical train loads is considered highly unlikely under normal operating conditions, this exclusion is justified. Finally, it should be noted that the SLS overload check required to use the improved fatigue strength achieved through the HFMI treatment is carried out using characteristic load combinations, as shown in Table 3.1.

Fatigue Limit State

FLS addresses the risk of fatigue failure caused by repeated cyclic loads throughout the design life of the bridge, which is a crucial factor for steel railway bridges. For fatigue assessment, the characteristic load combination is utilized in conjunction with specific fatigue load models, rather than the extreme factored loads used in ULS, shown in Table 3.1. Within the scope of this project, two methods were used for fatigue assessment: the equivalence stress range method (λ -method) and the damage accumulation method (ESR method). These methods are used because they are compatible with HFMI calculations.

3.3 Parametric Design and Optimization Framework

To implement structural analysis and prepare the model for optimization, a computational tool has been developed in MATLAB. The code features a modular structure and is controlled by a main script. This script defines all initial parameters, such as the geometry of the bridge, material properties, train load model, safety factors, and fatigue parameters. By centralizing these variables, a flexible model is created in which the input data can be easily adjusted, fulfilling a fundamental requirement for the subsequent optimization process.

The core of the parametric model is an optimization loop driven by a GA. By testing different combinations of input parameters (different "genomes"), the genetic algorithms continuously generate new solutions, while it always save the "elite" ones. To ensure that the generated solutions are static and physically achievable, an evaluation is carried out based on pre defined conditions and loads. This function checks not only the static checks such as ULS, SLS and FLS, but also some geometric constraints. The results of these checks are displayed as punishments on the "fitness value" of the current solution if the solution exceeds the constraints. The algo-

rithm applies a penalty score that increases in proportion to the extent to which the constraints is exceeded, and rejects the specific solution through this mechanism solution.

The valid solutions are then evaluated by the objective function (fitness function) which is defined by the chosen design parameters. In this model, the objective function is formulated to evaluate the economic efficiency of the design by minimizing the total costs. Total costs are calculated by adding together material costs, manufacturing costs (such as steel processing and welding), and surface treatment costs. Based on the cost evaluation, the GA iteratively improves the design until the optimal solution is found through reaching any of the termination criteria.

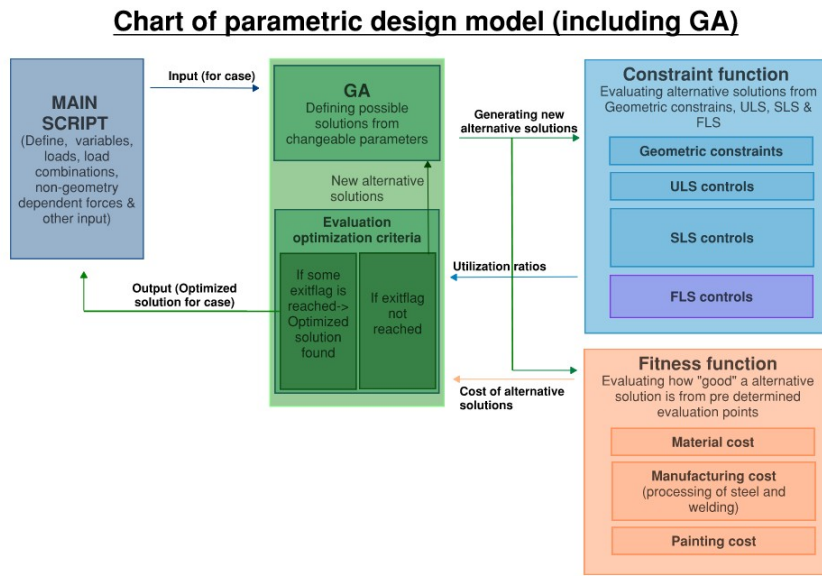


Figure 3.9: Chart over Parametric design model.

3.3.1 Details Subject to Fatigue

The fatigue details of the bridge was identified and evaluated according to the second generation of *Eurocode 3 Design of steel structures Part 1-9: Fatigue (second generation)* (Swedish Standards Institute, 2022). Fat-classes of treated details is based partially of Appendix F in the second generation Eurocode, and partially of recommendations from *International Institute Of Welding* (Marquis & Barsoum, 2016). Appendix F in the second generation Eurocode, is used for already defined HFMI-treatable details. While recommendations from *International Institute Of Welding* (Marquis & Barsoum, 2016), is used for HFMI-treatable details not yet defined in Eurocode. All details considered is shown in Figure 3.10. The characteristic value $\Delta\sigma_{C,HFMI,ref}$ is the value for a R-ratio of 0.1 and a steel class of S355 from tables presented in Appendix F in the second generation Eurocode.

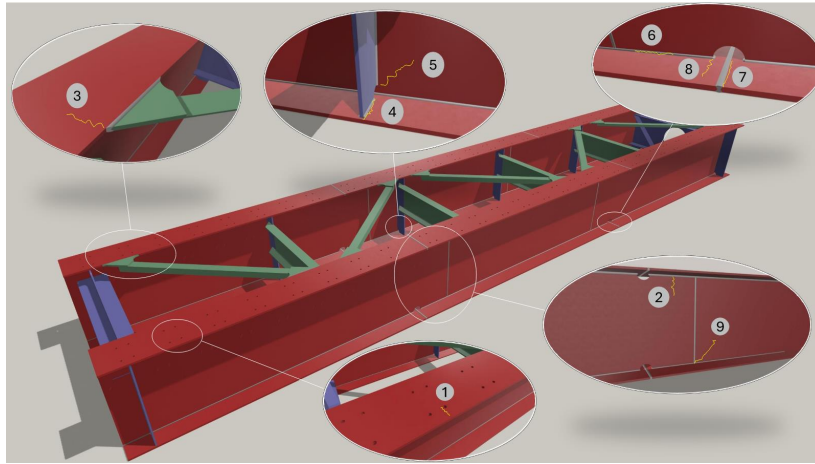


Figure 3.10: Overview of fatigue details on type bridge.

Detail 1: Hole in the upper flange (Sleeper attachment)

Description: Drilled holes in the upper flange for fastening the sleeper. Affected by normal stresses from global bending and normal forces, damage calculated with a slope of 5 until cut-off limit according to second generation Eurocode recommendations.

Detail category	Constructional detail
90 $m_1 = 5$	
50 $m_1 = 3$	

Figure 3.11: Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation), Detail 5, Table 10.2 (Mechanically fastened joints) (Swedish Standards Institute, 2022).

FAT class: AW FAT 90 ($\Delta\sigma$) (see Figure 3.11). Not applicable for HFMI treatment, stress ranges reduced since detail is in pure compression, according to Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation).

Detail 2: Longitudinal weld (Top/Bottom flange)

Description: The continuous longitudinal weld connecting the web and flanges. Evaluated for normal stresses resulting from global bending and normal force.

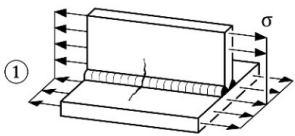
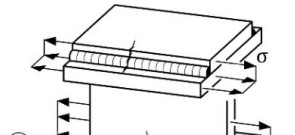
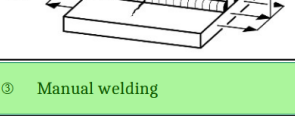
Detail category	Constructional detail
125	
112	
125	
112	
100	

Figure 3.12: *Eurocode 3: Design of steel structures - part 1-9: Fatigue*, Detail 3, Table 10.3 (Welded built-up sections and longitudinal welds) (Swedish Standards Institute, 2022).

FAT class: AW FAT 100 ($\Delta\sigma$) (see Figure 3.12) assuming manual weld (conservative for FAT-value), according to *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)*. Not applicable for HFMI treatment.

Detail 3: Attachment for L-profile (gusset plate)

Description: Welded gusset plate on the top flange of the main girder to connect the L-profiles of the bracing system. Evaluated for normal stresses resulting from global bending and normal force.

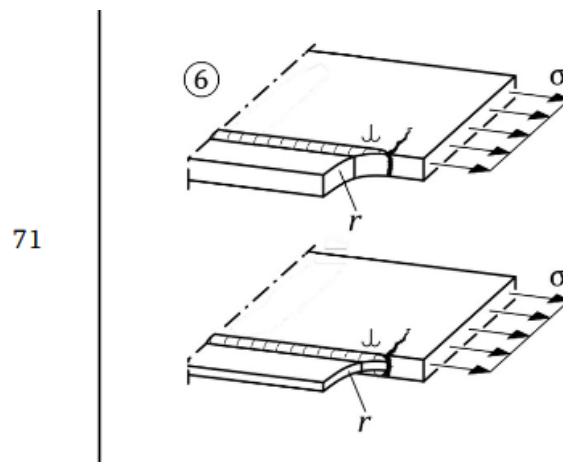


Figure 3.13: *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)*, Detail 6, Table 10.5 (Weld attachments and stiffeners) (Swedish Standards Institute, 2022).

FAT class: AW FAT 71 ($\Delta\sigma$) (See Figure 3.13). HFMI detail not yet fully confirmed and included in Eurocode. Research has been done and showing

some positive result on similar details, with an increase in FAT class of 5 classes (an increase with 5 FAT classes to FAT 90 has been achieved by Edgren et al., 2019, but for AW detail with FAT 50), which is assumed to be possible in this study and for this detail. An increase of 5 FAT classes is also in line with current recommendation according to IIW Marquis and Barsoum, 2016.

Detail 4: Transverse stiffener, weld toe at flange

Description: Stress concentration at the weld toe of the stiffener to flange connection. Evaluated for normal stresses resulting from global bending and normal force.

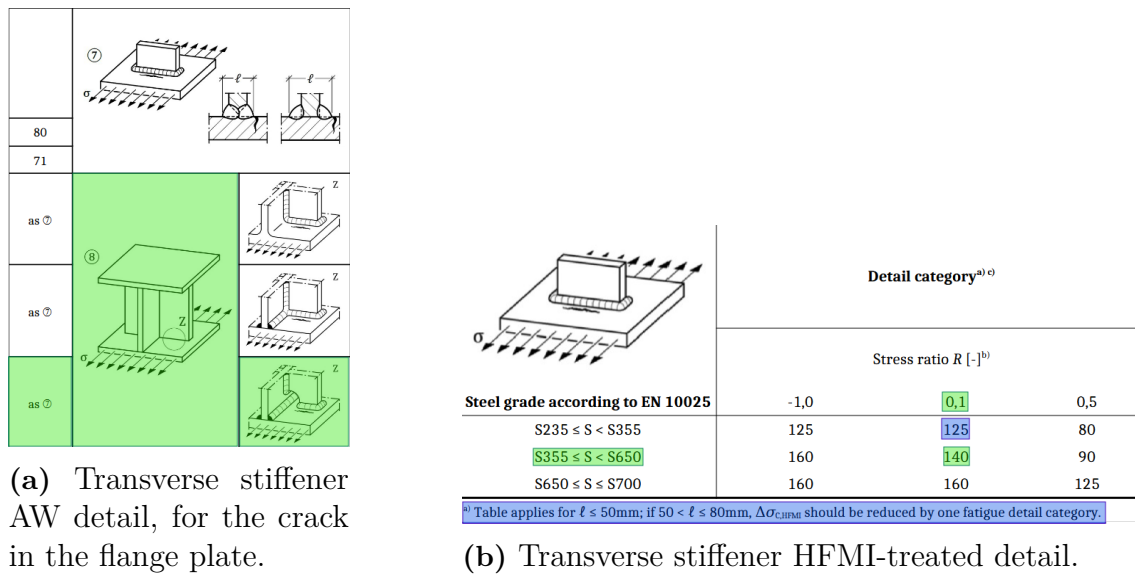


Figure 3.14: Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation), Detail 8, Table 10.5 (Weld attachments and stiffeners) (Swedish Standards Institute, 2022).

FAT class: AW FAT 71/80 ($\Delta\sigma$) (see Figure 3.14a) depending on the combined stiffener plate and weld thickness ($t \leq 50$ mm yields the higher FAT class). Can be HFMI-treated, increase to FAT 125/140 (see Figure 3.14b according to Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation) appendix F (Detail Transverse stiffener)).

Detail 5: Transverse stiffener, weld toe at web

Description: Stress concentration at the weld toe of the stiffener to flange connection. Evaluated considering principal stresses from global bending, normal force and shear according to Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation).

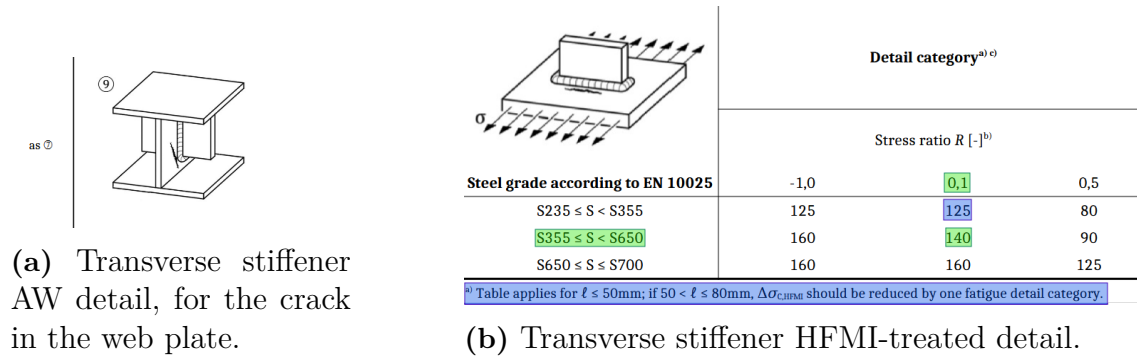


Figure 3.15: *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)*, Detail 9, Table 10.5 (Weld attachments and stiffeners) (Swedish Standards Institute, 2022).

FAT class: AW FAT 71/80 ($\Delta\sigma_p$) (see Figure 3.15a). Can be HFMI-treated, increase to FAT 125/140 (see Figure 3.15b) according to *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)* Appendix F (Detail Transverse stiffener).

Detail 6: Longitudinal weld (Shear stress)

Description: Shear in longitudinal weld connection bottom flange and web, evaluated considering shear stress from global shear force.

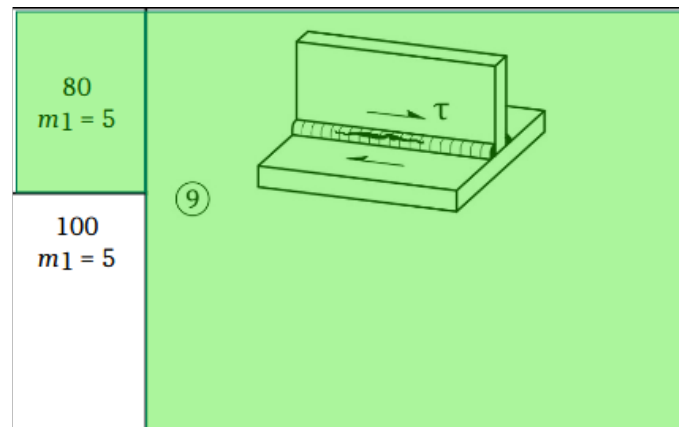


Figure 3.16: *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)*, Detail 9 Table 10.6 (Load carrying welded joints) (Swedish Standards Institute, 2022).

FAT class: FAT 80 ($\Delta\tau$) (see Figure 3.16). Not applicable for HFMI treatment.

Detail 7: Transverse butt weld in flange (Splice weld)

Description: Fully penetrated butt weld joining two flange plates (with or without taper, if tapered, stress concentration factor for thickness taper, k_f ,

is used), assuming an angle at weld toe of max 150 degrees (class B according to *ISO 6520-1*). Executed either in the factory or on-site dependent on maximum section length.

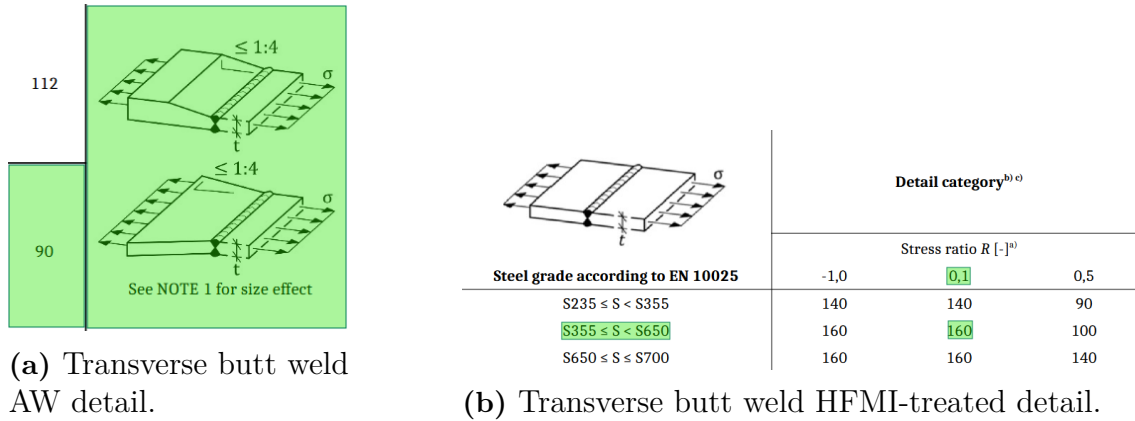
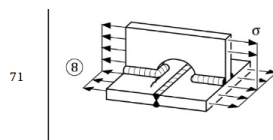


Figure 3.17: *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)*, Detail 19 (detail 19 is used instead of 18 to allow for HFMI treatment), Table 10.4 (Transverse welded butt joints (full penetration)) (Swedish Standards Institute, 2022).

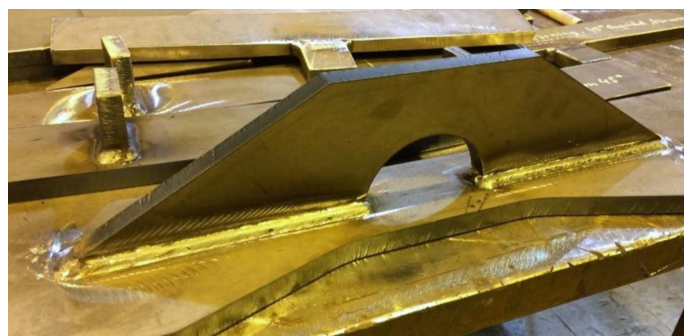
FAT class: AW FAT 90 ($\Delta\sigma$) (see Figure 3.17a). HFMI treatment could be used, upgraded to FAT 160 (see Figure 3.17b) according to *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)* appendix F (Transverse butt weld).

Detail 8: Weld toe at cope hole in web (Splice)

Description: Local stress concentrations at the weld ends around cope holes in the web, which are often required at transverse splices to ensure proper execution of the intersecting welds.



(a) AW detail for crack at cope hole in web plate.



(b) HFMI-treated detail used as sample in the study by Hedegard et al., 2020.

Figure 3.18: *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)*, Detail 8, Table 10.3 (Welded built-up sections and longitudinal welds) (Swedish Standards Institute, 2022).

FAT class: AW FAT 71 ($\Delta\sigma$) (see Figure 3.18a). Can be HFMI-treated, in project like Hedegard et al., 2020 this specific detail (see Figure 3.18b) has been showing positive response to both pure HFMI treatment and combinations of HFMI treatment and TIG-remelting on pre-fatigued details could result in a Fatigue strength of a corresponding new detail. In respect to those results and the current Eurocode guidelines is upgraded to FAT 100 according to *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)* appendix F, is assumed possible in this study. Longitudinal stiffener detail is used as a reference from Eurocode (corresponds to AW Detail 1 in Table 10.5 with $L \geq 200$ mm) which replaces Detail 8 in Table 10.3 for the case with $r_{cope} \leq 60$ mm.

Detail 9: Intersection of longitudinal and vertical weld in web (bi-axial weld)

Description: Connection between sections in web, where transverse vertical welds (splices) intersect with longitudinal welds. Subjected to bi-axial stress state, with welds following two axis intersecting in the same point. Evaluated for principal stresses at intersection of vertical and longitudinal weld.

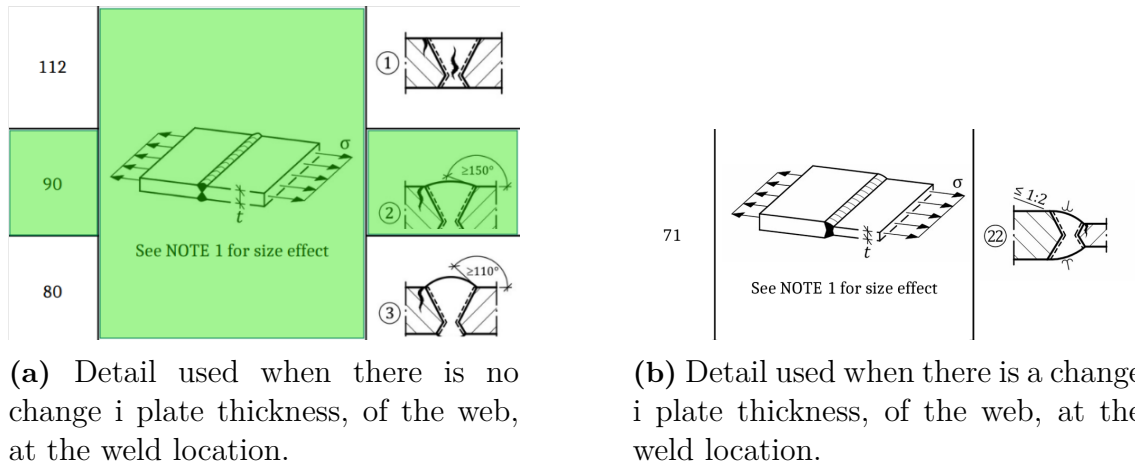


Figure 3.19: *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)*, Detail 2 & 22 (With and without change in plate thickness at the weld location), Table 10.4 (Transverse welded butt joints (full penetration)) (Swedish Standards Institute, 2022).

FAT class: No clear detail defined in for bi-axial intersecting welds *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)*, Detail 2 or 22 (dependent of if there is a change in plate thickness or not at the weld location), Table 10.4 (Transverse welded butt joints (full penetration)) is used. *Capacity:* AW FAT 90 or 71 ($\Delta\sigma_p$) (see Figure 3.19a or 3.19b). HFMI treatment could possibly be used. No clear current guidelines *Eurocode 3: Design of steel structures - part 1-9: Fatigue (second generation)* appendix F, in this study it is therefore assumed that this detail could be treated with a increase of 5 FAT classes, resulting in a increase to FAT 160 or 125 (depending on which AW FAT class that should be used), similarly to detail showcased in

Figure 3.17b, in accordance with IIW standards Marquis and Barsoum, 2016. The corresponding HFMI detail in Eurocode would be Transverse butt weld, but to confirm the possibility to actually treat this detail, further investigation is required. The HFMI-treated detail is also controlled for principal stresses at the point of the weld intersection.

3.3.2 Functions Used in PDM

To avoid combining all calculations into a single large script, the calculation process has been split into several separate functions. The main script invokes these functions sequentially to carry out specific tasks, such as calculating cross-section data, evaluating loads, or checking limit states. This modular structure makes the code easier to read, simplifies debugging, and makes the calculations more efficient, by minimizing the number of times each individual function needs to be invoked. Figure 3.20 presents a flowchart illustrating how the functions are connected, and Appendix B provides a brief description of each function's purpose and composition.

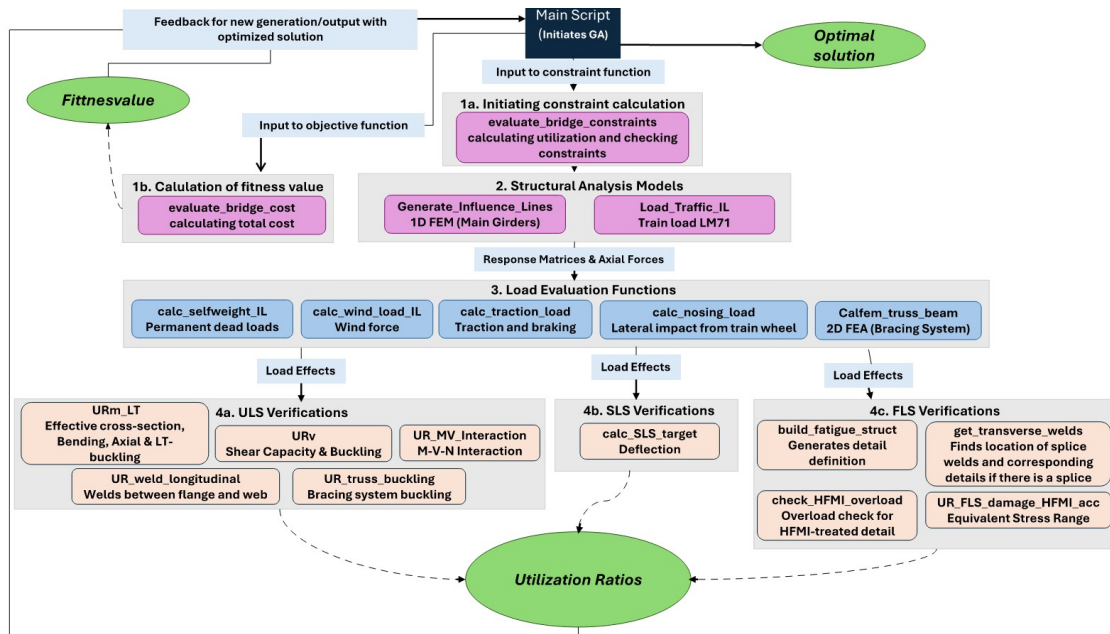


Figure 3.20: Flowchart that illustrates the connection of the functions.

3.3.3 Genetic Algorithm

To optimize the design of the bridge, a GA function is used from MATLAB's built-in optimization toolbox. A GA is a search and optimization technique in computer programming inspired by Charles Darwin's theory of evolution by natural selection (MathWorks, 2026). The method solves both linear and non-linear optimization problems in a stochastic manner by repeatedly modifying a population of individual solutions. It is therefore important to run the same type of simulation multiple times to ensure that the results are reliable. At each step, the algorithm selects the best members of the current population, elite individuals (based on their 'fitness

value') to reuse and combine them into new generations, in order to determine the best possible solution. The generation creation is done through operations such as crossover, mutation and random generation of new individuals within the defined limits. This allows for detection of global minimums for complex non-linear problem with large numbers of local minimums. As long as the populations size is large enough for the number of optimization variables, the population converges over time towards an optimal solution (MathWorks, 2026).

In this master thesis, GA is integrated with a constraint function, consisting of the evaluation functions described in Appendix B, and some geometric constraints. The algorithm adapts the combination of the optimization variables to find the most economical cross-sectional design, defined by the fitness function, without exceeding any of the constraints.

Although the optimization evaluates the total structural cost (see Figure 3.22), the material cost is one of the primary factors affected by the HFMI treatment and ultimately dictates the optimal solution. A decrease in material cost and material usage is also interesting considering environmental impact on steel structures. To consider this and also evaluate how HFMI treatment could be used to reduce carbon emissions from steel structures, some reference values representing "cradle to gate" CO_2 -values for hot-rolled plates with different yield strengths from Sperle et al., 2013 are utilized, as presented in Figure 3.21. Note that these values only will be used for presentation of carbon emissions but not used as an evaluation measurement in the optimization process.

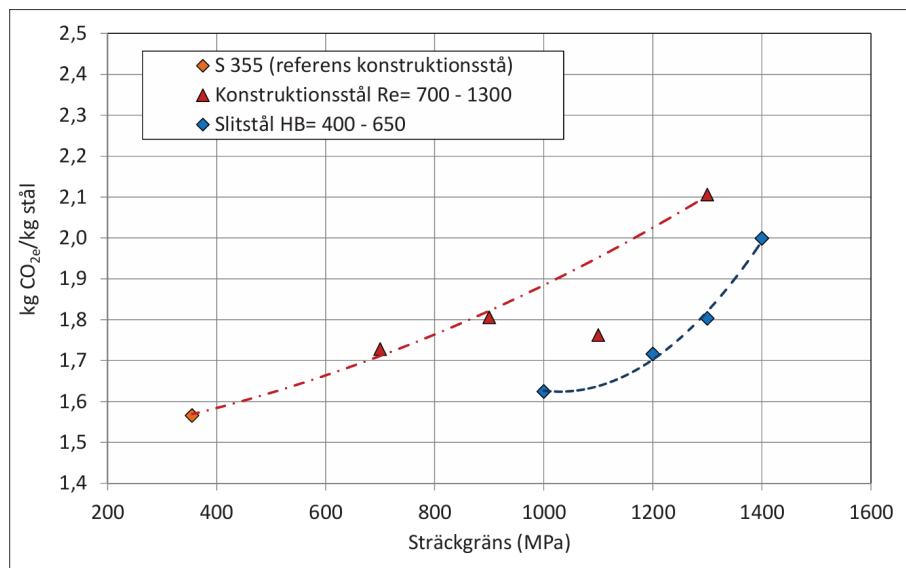


Figure 3.21: Carbon emission of raw material for hot rolled plates of different steel grades according to Sperle et al., 2013.

3.3.3.1 Constraints and Variables

To ensure that the optimization results in a bridge is realistic and possible to construct and in some manner reduce the computation time, a number of constraints and fixed parameters have been defined for the algorithm. These constraints are divided into hard constraints, which establish the absolute geometric boundaries of the search space (such as the total girder height limits of $L/15$ to $L/40$), and soft constraints, which represent the structural design checks that evaluate and govern the viability of the parameters. The soft constraints dictate what values they can be assigned in each specific case. While the rest of the parameters are all defined as discrete variables based of available products on the market and within dimensions that is assessed to be reasonable and practical for bridge construction. These are categorized as follows (see exact values in Table 3.5):

Geometry and Section Lengths:

- **Total Height:** The total height of the bridge can freely vary between $L/40$ and $L/15$, but as discrete points within the span. The $L/20$ to $L/30$ span is generally the 'standard' to originate from the bridge design (Collin et al., 2012), as they usually result in the most effective total height, under "normal" circumstances without restrictions on the geometry of the bridge. But to allow the model to freely find the optimal solutions with the boundaries and evaluation included the height is allowed to move outside those standard guidelines.
- **Width of Flanges:** The minimum width of the flanges is defined to provide enough space to fit the boltholes of the sleepers, in accordance with *Eurocode 3: Design of steel structures - Part 1-8: Design of joints* (Swedish Standards Institute, 2005b). Although this requirement only applies to the top flanges, the minimum value is adopted for both. The maximum width is determined by the bridge's geometry to prevent any theoretical intersection between the main girders (ensuring, in practice, a minimum clearance of 50 mm between the flanges of the two girders).
- **Section Lengths:** There is a possibility of change in section along the bridge. At maximum there could be 5 sections along the beam but they are always symmetric in longitudinally, which result in maximum 3 different section geometries. The individual sections cannot be longer than 12 m (a production restriction based on the maximum length available in storage at supplier) and a soft lower threshold value based of manufacturing and welding cost.
- **Number of Splices:** Number of splices (n_{splice}) are allowed to change from 0, 2, or 4 with the assumption of splices always being placed symmetric in longitudinally. Bridges shorter than or equal to maximum section length could have 0, 2 or 4 splices, bridges between maximum section length and 3 times maximum section length could have 2 or 4 splices, bridges longer than 3 times maximum section length will have 4 splices, with all cases controlled by the cost of splices as a soft limit.

- **Distance Between Transverse Cross-Beams (a_1):** The possible centre to centre distance for cross-beams is set to be between 2000 and 6000 mm. The distance can vary in increments that ensure there is always an even number of "bays" along the full length of the beam.

Plate Thicknesses and Materials:

- **Standard Dimensions:** To reduce manufacturing costs and avoid custom orders, the algorithm is constrained to selecting only plate thicknesses that are available as standard from the supplier Tibnor, n.d. This also significantly reduces the number of variables within the algorithm. In addition to this the upper boundary for plate thickness is set to 60 mm, since both welding cost increases and available specifications of steel in certain dimensions decreases above this limit.
- **Thickness Restrictions (both flanges and web):** There is no hard limit for plate thickness other than the values accessible from suppliers and the upper limit. However, soft constraints are introduced through factors accounting for the thickness effect on structural capacity, as well as the increased cost of thicker (and thinner than 10 mm) plates according to SSAB, 2026 add on price estimate.
- **Transverse Stiffener Plates:** The thickness of the transverse stiffener plates is limited to 10 mm, while their width is fixed relative to the flange width (always terminating 15 mm from the edge of the narrower flange).

Profiles:

- **Hot-Rolled Profiles:** UPE-beams and L-profiles are used as a stiffening truss system between top flanges, restricting the critical length of lateral torsional buckling. The size of the profiles is constrained to vary between the available sizes from (Tibnor, n.d.). The placement of profiles is determined by the a_1 -measurement, as previously mentioned.

Detailed Design and Joints:

- **Transverse Splices:** Changes in plate thickness and width at transverse splices along the beam are assumed to be achieved by tapering the thicker or wider plate with a 1:4 slope, fulfilling the fatigue detailing requirements specified in *Eurocode 3 Design of steel structures Part 1.9: Fatigue* (Swedish Standards Institute, 2022). The thickness taper of the web plate is executed symmetrically with respect to the beam's centerline, whereas the flange plates are always tapered towards the web to ensure flush top and bottom flange surfaces. Thickness variations are governed by soft constraints that account for the cost of milling the tapers and the stress concentration factors associated with the asymmetric flange tapers, which lead to higher fatigue stresses in the transverse welds. Furthermore, the web splice is consistently located 500 mm away from the flange splice, offset towards the midspan of the beam.

- **Throat Thickness of Longitudinal Fillet Weld:** The throat thickness of the fillet weld is allowed to vary between some discrete values, with 3 mm as a lower bound according to *Eurocode 3: Design of steel structures - Part 1-8: Design of joints* (Swedish Standards Institute, 2005b). It is governed by the increasing cost for larger weld cross-section, the ULS and FLS capacity.

Table 3.5: Variable values used in PDM.

Variable Category	Notation	Value Range / Discrete Options	Limiting Factors / Constraints
Flange Widths	b_{fo}, b_{fu}	$250 \text{ mm} \leq b_f \leq (C_c/2 - 50) \cdot 2$ (Increments of 10 mm)	Min: < Bolthole spacing for sleepers. Max: 50 mm clearance between girders to avoid intersection.
Plate Thicknesses	t_{fo}, t_{fu}, t_w	[8, 10, 12, 15, 20, 25, 30, 35, 40, 45, 50, 60] mm	Min/Max: Standard availability from suppliers. Soft limits via cost penalties for thicker plates and thickness effect on capacity.
Total Height	H_{tot}	$L_{beam}/40 \leq H_{tot} \leq L_{beam}/15$ (Discrete points, 25 total steps)	Min/Max: Exceeding what is 'standard' guidelines slightly.
Number of Bays (Cross-beams)	n_{fack}	Derived from $2000 \leq a_1 \leq 6000 \text{ mm}$. $n_{min} \leq n_{fack} \leq n_{max}$	Min/Max: Must result in an even number of bays along the full span length.
Splice Locations	LOC	Max section length = 12 m.	Max: Supplier storage/-transport length restrictions. Min: Soft limit based on manufacturing/welding cost.
Number of Splices	n_{splice}	[0, 2, 4] (Depending on bridge length)	Limit: Geometrical necessity based on max section length vs. total span, and cost penalties.

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Table 3.5 – Continued from the previous page

Variable Category	Notation	Value Range / Discrete Options	Limiting Factors / Constraints
Longitudinal Fillet Weld	a_{bw}	[4, 5, 6, 7, 8] mm	Min: Minimum allowed throat thickness according to <i>Eurocode 3: Design of steel structures - Part 1-8: Design of joints</i> and ULS and FLS capacity. Max: Soft limit via increased cost for larger weld volumes.
Truss Profiles	UPE, L-profile	Discrete standard sizes available from supplier (Tibnor, n.d.)	Min/Max: Standard availability from suppliers.

3.3.3.2 Fitness Function and Optimization Criteria

The objective of the optimization is to minimize the total structural cost (including what are considered to be the main parameters affecting manufacturing cost) while maintaining structural integrity across all limit states (ULS, SLS, and FLS). The cost function, C_{total} , is discretized into four primary domains, as illustrated in Figure 3.22. The precise geometric quantification of all manufacturing activities is summarized in Table 3.8.

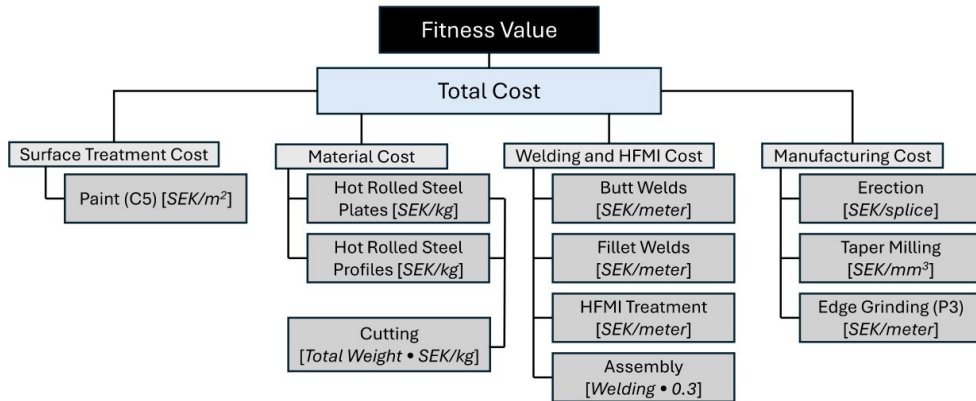


Figure 3.22: Flowchart of which costs that are included to calculate the fitness value in the objective function used for optimization.

Table 3.6: Cost quantification for material, welding, and surface treatment.

Category	Specification	Steel Grades / Cost			Unit	Comment
<i>Material Cost (Hot Rolled)</i>						
Component	Thickness (t)	S355	S460	S690		
Steel plates	$t \leq 60$ mm	10.23	13.00	15.73	SEK/kg	Given value after contact with SSAB.
Steel profiles	-	16.37	20.80	25.17	SEK/kg	Based on the fact that steel sections have a factor of 1.6 compared to steel plates, according to (Hlal, 2025).
<i>Welding and HFMI Cost</i>						
Standard welding	Conventional fillet and butt welds	See Equation 3.6			SEK/m	Reference prices used in Equation 3.6 (value for S355 from Hlal, 2025).
Edge preparation of butt weld	Thickness $t > 20$ mm	350	350	350	SEK/m	Edge preparation according to SS-EN ISO 9692-1 for double V or K weld (Hlal, 2025).
Edge preparation of butt weld	Thickness $t \leq 20$ mm	200	200	200	SEK/m	Edge preparation according to SS-EN ISO 9692-1 for double V or K weld (Hlal, 2025).
HFMI treatment	Factory	187	187	187	SEK/m	Calculated according to values from Englund and Dagduk, 2022, see Table 3.7.

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Table 3.6 – Continued from the previous page

Category	Specification	Steel Grades / Cost			Unit	Comment
HFMI treatment	On-site	280.5	280.5	280.5	SEK/m	Factory value multiplied with a factor of 1.5 according to Neher, 2026.
Assembly	Factory	Welding Cost $\times$ 0.3			-	Factor considering assembly in workshop, moving part, spot welding Etc., using a surcharge of 30% of the welding cost (Hlal, 2025).
<i>Surface Treatment Cost</i>						
Painting	Factory	1000	1000	1000	c_{paint} SEK/m^2	Painting that fulfils C5 standard, reference value from Hlal, 2025.
Painting	On-site	3000	3000	3000	c_{paint} SEK/m^2	Painting that fulfils C5 standard, reference value from Hlal, 2025.
<i>Manufacturing and Machining Cost</i>						
Machining / Milling	Thickness Ta- per (1:4 Slope)	0.001	0.001	0.001	SEK/mm^3	Estimated value with help of AI
Edge grinding (P3)	Grinding of edges	125	125	125	SEK/m	Reference value from Hlal, 2025.
Erection	On-site splice weld	9000	9000	9000	$SEK/splice$	Cost for splice welds done on-site (n_{splice}), reference value from Hlal, 2025.

Material Cost

Structural steel costs are evaluated based on mass, accounting for both yield strength (f_y) and the change in price for both thickness and width of plates. A distinction is made between hot-rolled profiles (UPE and L-sections) and hot-rolled heavy plates.

For plates a base unit price is used (see Table 3.6) dependent on steel grade, which extra cost is added to depending on dimensions according to SSAB, 2026.

$$C_{material} = \sum (V_i \cdot \rho_s \cdot C_{type,grade,t}) \quad (3.4)$$

Where $C_{type,grade,t}$ is the unit price (SEK/kg) which varies dynamically with the steel grade according to Table 3.6.

Welding and HFMI Cost

Welding costs are modelled as a function of the number of passes (weld strings) required for a specific weld cross-section (Equation 3.5) and hourly labour/material cost ($c_{ref, weld, grade}/4$) according to Hlal, 2025, with an increase in hourly cost dependent on steel grade (estimated increase in hourly cost, from slower welding speed and increased material cost with the help of AI). Welds that are considered is longitudinal flange to web welds, welds attaching transverse stiffener plates and splice welds (see Table 3.8).

- **Butt Welds:** Modeled as double V-weld or K-weld dependent on specific weld location preparations with a 60° opening angle and a 1.5 mm root gap is assumed. An edge preparation cost (machining the edge to correct angle) is applied dependent on volume of removed material and weld type, since double V welds have two edges that needs preparation and a K-weld only has one.
- **Fillet Welds:** Calculated based on the effective throat thickness (a_{weld}).
- **HFMI Treatment:** HFMI treatment is applied to critical fatigue details where it is applicable, including transverse splices, cope holes, attachment for L-profiles and transverse stiffener plates. The unit cost (SEK/m) estimated with values from Englund and Dagduk, 2022 covering cost of equipment rental, labour cost and some added cost for quality assurance (see Table 3.6).

$$n_{passes} = \frac{A_{weld} + 30}{19} \quad (3.5)$$

$$C_{weld} = n_{passes} \cdot \frac{C_{labour, weld}}{v_{weld}} \quad (3.6)$$

$$C_{prep, butt} = \begin{cases} 200 \text{ SEK/m,} & \text{if } t_i \leq 20 \text{ mm} \\ 350 \text{ SEK/m,} & \text{if } t_i > 20 \text{ mm} \end{cases} \quad (3.7)$$

$$C_{weld} = L_i \cdot C_{weld} + L_i \cdot C_{prep.butt} \quad (3.8)$$

$C_{labour, weld}$ are the labour cost of welding, 550 SEK/hour (Hlal, 2025).

v_{weld} are the welding speed 4 m/hour.

Table 3.7: Data used for calculation of an per meter cost of HFMI treatment "On-site" and "in factory".

Parameter	Value	Comment
<i>Input Parameters</i>		
Labour cost	500 <i>SEK/h</i>	Used in similar study by Englund and Dagduk, 2022.
Treatment speed	15 <i>m/h</i>	Used in similar study by Englund and Dagduk, 2022.
Leasing of equipment	1200 <i>SEK/day</i>	Used in similar study by Englund and Dagduk, 2022.
Working time per day	6 <i>h/day</i>	Used in similar study by Englund and Dagduk, 2022.
Safety/margin factor for quality assurance	4	Used in similar study by Englund and Dagduk, 2022.
Factor for on-site work	1.5	Value selected based on email communication with Neher, 2026.
<i>Calculated Costs</i>		
Factory	187 <i>SEK/m</i>	Calculated based on inputs above.
On site	280.5 <i>SEK/m</i>	Calculated based on inputs above.

Table 3.7 presents the data used to calculate the cost per meter (*SEK/m*) for HFMI treatment, based on key industry values and a similar study by Englund and Dagduk, 2022. For calculation of "On-site" per meter cost for HFMI treatment (*SEK/m*) an increase of 15-50%, on top of "in factory" values was suggested by Neher, 2026. In this study a factor of 1.5 (50% increase in cost) was chosen to be used, to ensure a "conservative" estimate of the cost savings associated with HFMI treatment "on-site".

Surface Treatment Cost:

Corrosion protection (blasting and painting) is calculated per square meter of surface area, using a reference value, for C5 standard, from Hlal, 2025. Category C5, as defined in the *EN ISO 12944-2* standard, describes environments with very high corrosivity that require the use of long-lasting coating systems (Swedish Standards Institute, 2017). The area considered is the area of main girders and transverse stiffener plates (see Table 3.8). To account for the increased costs associated with on-site painting of welded or HFMI-treated details, a higher reference price for surface treatment is applied (see Table 3.8). On site painting is only considered for the transverse splices with a length of 500 mm from the weld in both directions. This length is based of the required length to HFMI treat in each direction from the splice

cope hole according to Englund and Dagduk, 2022.

$$C_{paint} = A_{mantle} \cdot C_{factory} + A_{site} \cdot C_{site} \quad (3.9)$$

Where:

- C_{paint} is the total cost of the surface treatment (SEK)
- A_{mantle} is the surface area painted at the factory (m^2)
- $C_{factory}$ is the unit cost for factory surface treatment (SEK/ m^2)
- A_{site} is the surface area painted on-site (m^2)
- C_{site} is the unit cost for on-site surface treatment (SEK/ m^2)

Manufacturing and Machining Cost:

The model accounts for manufacturing costs using key values derived from previous research, industry data, and artificial intelligence and are summarized in Table 3.6. The assumptions and methodologies for these costs include:

- **On-site Welded Splices (Erection):** An additional cost of 9,000 SEK per splice is applied to account for the increased preparation and setup time and for the on-site transverse butt weld, as suggested by a similar analysis conducted by Hlal, 2025.
- **Taper Milling:** Volumetric milling/grinding of a 1:4 slope at thickness transitions to satisfy Eurocode fatigue requirements. The milled volume is based on the width of the thicker plate, and the milling cost is estimated at 0.001 SEK/ mm^3 . This value was calculated based of labour cost and estimated time of conducting such a work, with the help of AI. The used value is from the upper part of the estimated possible range of cost.
- **Edge Grinding:** Mechanical grinding of the outstanding longitudinal girder edges to satisfy the P3 surface preparation standard according to *ISO 8501-3:2025*. The reference price used is based on a similar analysis conducted by Hlal, 2025.
- **Cutting and Assembly:** Plate cutting costs are based on total weight, while general workshop assembly is estimated as a 30% surcharge on total welding cost as suggested by industry practice (Hlal, 2025).

Table 3.8: Geometric quantification of material and manufacturing operations.

Component / Activity	Calculation Method (per unit)	Total Quantity / Multiplier
<i>Material Volumes</i>		
Main Girder Plates	$V_{sec} = L_{sec} \cdot (t_{tf}b_{tf} + t_{bf}b_{bf} + t_w h_w)$	Evaluated per section $\times$ 2 girders
Truss Cross-Beams	$L_{UPE} = C_c - t_w - 2b_{stiff}$	$n_{bays} + 1$
Truss Diagonals	$L_{diag} = \sqrt{a_1^2 + W_{gap}^2}$	n_{bays} (1 per bay)
Transverse Stiffeners	$V_{stiff} = b_{stiff} \cdot t_s \cdot h_w$	$\times 2$ (plates per cross-section) $\times (n_{bays} + 1)$

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3. Methods

Table 3.8 – Continued from the previous page

Component / Activity	Calculation Method (per unit)	Total Quantity / Multiplier
<i>Welding Parameters</i>		
Longitudinal Bottom (Fillet Weld)	Length: L_{bridge} Area: $A_{weld} = a_{bw}^2$	$\times 4$ (both web faces, 2 girders)
Longitudinal Top (K-Butt Weld)	Length: L_{sec} Area: $A_{weld} = \frac{t_w^2}{4} \tan(30^\circ) + 1.5t_w$	$\times 2$ (2 girders)
Transverse Stiffeners (Fillet Weld)	Length: $L_{weld} = 2b_{stiff} + h_w - 2r_{cope}$ Area: $A_{weld} = a_{stiff}^2$	$\times 4$ (double-sided, 2 plates) $\times (n_{bays} + 1)$
Transverse Splices (Double-V Butt Weld)	Length: $L_{splice} = (w_i + w_{i+1})/2$ Area: $A_{weld} = \frac{t_{avg}^2}{2} \tan(30^\circ) + 1.5t_{avg}$	$\times 2$ (2 girders) $\times n_{splice}$
<i>Surface Treatment</i>		
Painting (Mantle)	$A_{mantle} = (2b_{tf} + 2b_{bf} + 2h_w) \cdot L_{sec} - A_{site}$	Evaluated per section $\times 2$ girders
Painting (Stiffeners)	$A_{stiff} = b_{stiff} \cdot h_w$	$\times 4$ (2 plates, 2 faces) $\times (n_{bays} + 1)$
Site Painting (Splices)	$A_{site} = 2 \cdot L_{splice} \cdot 1.0 \text{ m}$	$\times 2$ (2 girders) $\times n_{splice,site}$
<i>Machining and Logistics</i>		
Taper Milling	$V_{taper} = 2 \cdot (\Delta t)^2 \cdot w_{thicker_plate}$	Evaluated at every thickness jump
Edge Grinding (P3)	Full bridge length, L_{bridge}	$\times 8$ outer edges per girder $\times 2$ girders
Plate Width Order	Fixed cost per unique width b_i	n_{unique_widths}

4

Results

The results presented in this chapter are derived from the developed MATLAB script, which forms the basis for addressing the research questions of this master's thesis. To verify the reliability of the PDM, its output has been compared with hand calculations (detailed in Appendix A) and the design documents. Following this verification, this chapter presents the structural utilization rates for span lengths of 10, 20, 30 and 40 m, and HFMI-processed configurations are compared for different steel grades with traditional AW designs. The chapter concludes with a comparative analysis of the material savings, economic benefits and environmental impact. In all optimizations result presented in this chapter, possible variables of optimization are presented in Table 4.1.

Table 4.1: Spans of design variables used in optimization. The design vector contains 23 variable values in total, variables marked (3×) are repeated for the three symmetric half-sections of the bridge, and (2×) for the two splice locations.

Dimension	Symbol	Range / discrete set
Top-flange thickness (3×)	t_{fo}	8,10,12,15,20,25,30,35,40,45,50,60 mm
Top-flange width (3×)	b_{fo}	250–1600 mm, step 10 mm
Web thickness (3×)	t_w	8,10,12,15,20,25,30,35,40,45,50,60 mm
Total girder height	H_{tot}	$L/40 - L/15$, 25 steps (rounded to 10 mm)
Bottom-flange thickness (3×)	t_{fu}	8,10,12,15,20,25,30,35,40,45,50,60 mm
Bottom-flange width (3×)	b_{fu}	250–1600 mm, step 10 mm
Number of bracing bays	n_{bays}	$L/2000 - 1$, $L/2000$
Bottom longitudinal weld throat	a_{bw}	4,5,6,7,8 mm
Splice locations (2×)	c_1, c_2	500 mm to $L/2$, step 500 mm
Cross-beam UPE profile	id _{UPE}	UPE140 to UPE400 (11 sizes)
Diagonal L profile	id _L	L80×8 to L200×18 (23 sizes)
Number of splices	n_{splice}	1, 2, 3 (0, 1 or 2 splices per half)

4.1 Verification on PDM

To be sure that the final results MATLAB provides is correct, it is essential to validate the developed and used tools and models. Therefore, output values from PDM is compared with hand calculations, and for some values with calculations from already verified work, in Table 4.2. By evaluating the percentage difference between these sources, the margin of error is ensured to remain within acceptable limits for the remaining work. In Tables 4.2 and 4.3 the "Difference" column shows the absolute mathematical difference between the calculated results and their corresponding verification values. The ESR function is verified using a known existing example from Al-Emrani and Ayg  l, 2014, using both a independent model and hand calculations, with values presented in Figure 4.2 and Table 4.4.

The verification covers a total of 12 different "cases" (combinations of span lengths, steel grades and AW or HFMI-treated) comparing the PDM values with verification values, and 10 different "cases" comparing the hand calculation with design calculation of the initial bridge. Both including verification of overall controls of the structural behaviour (e.g. bending and global stability) and local phenomenon such as patch load capacity. In addition, key parts of the cross-section, such as upper and lower longitudinal welds, UPE cross-beams, and L-diagonals, are individually evaluated. Finally, the resulting values for the SLS (deflection) and FLS are calculated and compared between the models. It should be noted that the natural frequency is also extracted and compared to demonstrate awareness of the phenomenon, although it does not act as an active design constraint in the PDM. The fatigue comparison is carried out using both the λ -method and the ESR method. These fatigue evaluations are only compared between the PDM and the hand calculations, for the λ -method and using the previously mentioned example for the ESR function.

Table 4.2: Verification of results comparing PDM and verification sources.

PDM	Verification value	Difference	Verification source
<i>Case 1: Bending UR</i>			
98.68%	99.83%	1.15%	Hand calculation
<i>Case 2: Global stability (value > 1 ok)</i>			
1.24	1.17	0.07	Hand calculation
<i>Case 3: Local stability</i>			
45.5%	46.7%	1.2%	Hand calculation
<i>Case 4: Shear capacity</i>			
44.08%	43.39%	1.31%	Hand calculation

Continue on the next page...

Table 4.2 – Continued from the previous page

PDM	Verification value	Difference	Verification source
<i>Case 5: ULS max UR for top longitudinal weld</i>			
31.94%	31.4%	0.5%	Hand calculation
<i>Case 6: ULS max UR for bottom longitudinal weld</i>			
46.9%	46.16%	0.83%	Hand calculation
<i>Case 7: ULS local web patch load capacity</i>			
12.47%	12.6%	0.13%	Hand calculation
<i>Case 8: ULS combined yielding</i>			
66.1%	88.59%	22.49%	Hand calculation
<i>Case 9: Max UR for UPE cross-beams</i>			
4.03%	2.25%	1.78%	Hand calculation
<i>Case 10: Max UR for L-diagonals</i>			
67.22%	56.72%	10.5%	Hand calculation
<i>Case 11: Deflection SLS</i>			
16.0 mm	16.01 mm	0.01 mm	Hand calculation
<i>Case 12: Eigenfrequency</i>			
19.72 Hz	19.66 Hz	0.14 Hz	Hand calculation
<i>Case 13: Fatigue with λ-method</i>			
74.11%	72.5%	1.61%	Hand calculation
106.67%	104.53%	2.14%	Hand calculation
151.47%	155.33%	3.86%	Hand calculation
129.21%	128.28%	0.93%	Hand calculation
117.81%	118.67%	0.86%	Hand calculation
59.46%	57.62%	5.57%	Hand calculation
99.1%	91.0%	8.1%	Detail 1, Note 1
90.5%	99.0%	8.5%	Detail 2, Note 1
73.0%	73.0%	0.0%	Detail 3, Note 1

Note 1: Fatigue design of steel and composite bridges Example 4.2 (Al-Emrani &

Aygül, 2014).

To ensure that the hand calculations are reliable for verifying the PDM model, the hand calculations have been compared with the initial calculations of the type bridge (see Table 4.3). The difference is due to the hand calculations and the PDM model being designed as hinged connections, which results in lower forces compared with the initial calculations, which used FEA.

Table 4.3: Verification of hand calculation results compared to design documents.

Hand calculation	Verification value	Difference	Verification source
<i>Case 1: Bending UR</i>			
99.83%	99.5%	0.33%	Design documents
<i>Case 2: Lateral torsional stability (>1 ok)</i>			
1.17	1.07	0.1	Design documents
<i>Case 3: Capacity for the vertical stiffeners</i>			
20.4%	20.5%	0.1%	Design documents
<i>Case 4: Shear capacity</i>			
43.39%	45.6%	2.21%	Design documents
<i>Case 5: ULS max UR for top longitudinal weld</i>			
31.4%	-%	-%	-
<i>Case 6: ULS max UR for bottom longitudinal weld</i>			
46.16%	48.56%	2.4%	Design documents
<i>Case 7: Max UR for UPE cross-beams</i>			
2.25%	6.91%	4.66%	Design documents
<i>Case 8: Max UR for L-diagonals</i>			
56.86%	93.83%	36.97%	Design documents
<i>Case 9: ULS local web patch load capacity</i>			
12.6%	- %	- %	-
<i>Case 10: Deflection SLS</i>			
16.01 mm	15.5 mm	0.59 mm	Design documents

Due to the lack of a simple hand calculation procedure for the ESR method, verifi-

cation was conducted by comparing hand calculated values against an independent tool. This verification process utilizes an example from Section 4.2.3 in Al-Emrani and Ayg  l, 2014, *Fatigue design of steel and composite bridges*, to validate the ESR-function developed for the PDM. The stress history generated by the PDM function is shown in Figure 4.1. Furthermore, the summation of stress cycles was verified against values from an independent source, with both stress histograms presented in Figure 4.2. Since nearly identical results were generated by the two independent sources, this confirm the validity of the stress generation and summation.

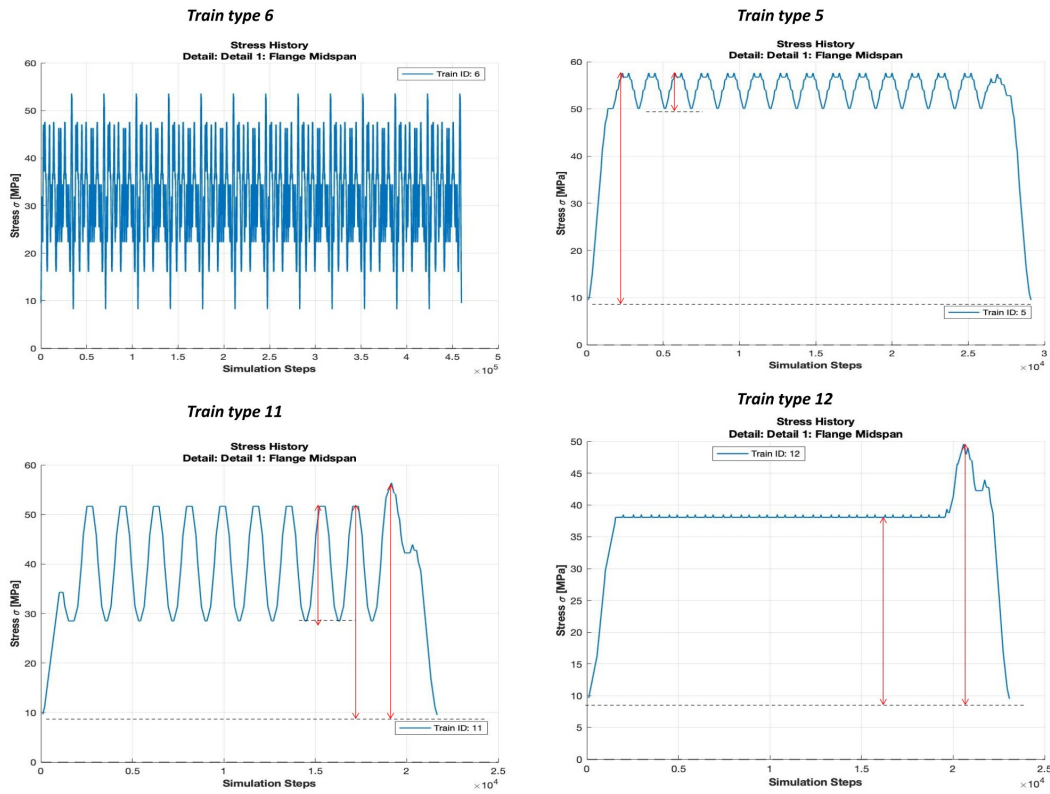
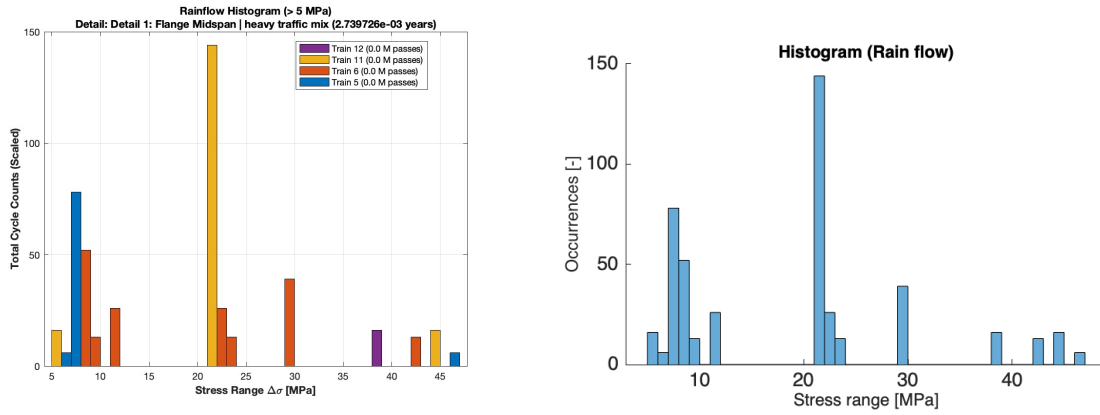


Figure 4.1: Stress history with train 5, 6, 11 & 12, from the developed PDM model.

To further verify the actual damage calculation of the function developed for the PDM, the details from the previously mentioned example were used, together with the stress cycles from the independent tool that generated the stress histogram in Figure 4.2b. These stress cycles were used to manually calculate the damage for the three details from the example and the complete calculations are provided in Appendix C. In Table 4.4, values from both the hand calculations and the PDM function are presented, with the PDM values being slightly lower for all details. This variation is attributable to the specific rainflow counting algorithms employed. The independent verification tool relies on an older model by Downing and Socie, 1982, which simplifies the process by counting half-cycles as full cycles. Conversely, the PDM utilizes a rainflow counting function from The MathWorks, Inc., 2026, based on ASTM International, 2017, which distinguishes between closed (full) and unclosed (half) cycles. This explains the slightly higher damage estimates yielded by the hand calculations.



(a) Stress histograms, using ESR-function developed for PDM, for one day and Heavy mix for detail 1 from the example in chapter 4.2.3 by Al-Emrani and Aygöl, 2014.

(b) Stress histograms, independent tool to calculate and summed up stress history, for on day and Heavy mix for detail 1 from the example in chapter 4.2.3 by Al-Emrani and Aygöl, 2014.

Figure 4.2: Stress cycles generated and summed up using Heavy mix from *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003), for the time period one day.

Table 4.4: Comparison in accumulated damage of details from example in Al-Emrani and Ayg  l, 2014, comparing hand calculated values with PDM.

Description	Detail 1	Detail 2	Detail 3	Comment
PDM: damage with ESR-method	0.4325	0.7267	0.17	Using rainflow counting by The MathWorks, Inc., 2026, and developed ESR function used in PDM.
Hand-calculation: damage with ESR-method	0.46	0.74	0.19	Using stress ranges and cycle counting from No2Fat. Hand-calculated damage in excel (See Appendix C)
Difference	6.33%	1.51%	11.07%	Difference between damage from PDM and hand calculations made in excel (See Appendix C)

4.2 Optimization Results

This section presents a summary of the optimization results for all investigated span lengths (10, 20, 30 and 40 m) for both AW and HFMI-treated states using steel grade S355 for AW and S355, S460 and S690 with HFMI treatment. The aim is to demonstrate how the choice of material grade and usage of HFMI affects the limiting state, costs, and material usage for different span lengths. An important statement for all of the result chapter is that a reduction in cost, material, or CO_2 -emissions is defined as a positive percentage change, while an increased cost is shown as a negative percentage change, as seen in e.g. Figure 4.3.

The general trends of the potential savings from usage of HFMI combined with different steel grades are shown in Figure 4.3, with all HFMI-treated states being compared with the AW state using only S355, as it is used as the "standard case". As shown in the Figure 4.3, the general result shows that there is potential savings in total cost using HFMI combined with S355 for spans up to about 34 m (linearly interpolated between 30 and 40 m), while HFMI combined with S460 could have some potential for shorter spans and S690 always being more expensive than the AW case using S355. To illustrate more clearly how the optimized cross-section changes along the bridge result in these findings, and to analyse why the HFMI treatment might offer the observed benefits, a detailed, section-by-section analysis was carried out for the 20 m bridges. These detailed and presented results are shown in the following Sections (4.2.1-4.2.3).

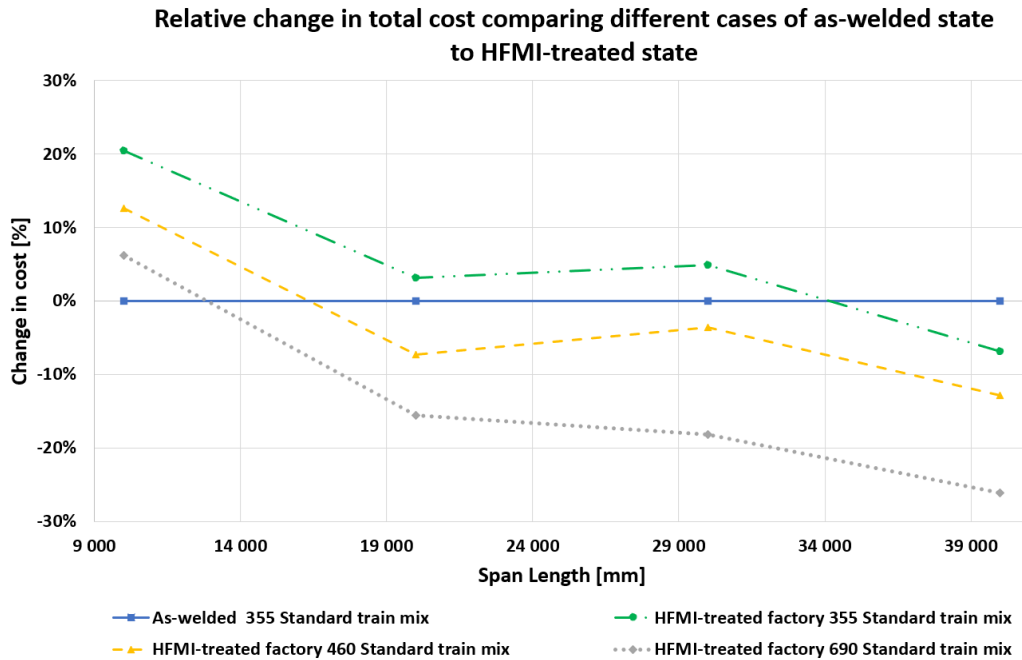


Figure 4.3: Relative change in cost comparing HFMI-treated state, using different steel grades for span lengths from 10 m to 40 m, to AW state using S355.

The governing details for the different studied cases in both AW and HFMI-treated

conditions are presented in Table 4.5, with the same detail numbering as used in Figure 3.10. The results in this table indicate that in the AW condition, detail 3 (gusset plate used for attaching the L-profiles) and detail 4 (transverse stiffener plate) are the critical ones for all studied span lengths. For the HFMI-treated case using S355 and shorter spans, the critical detail becomes detail 2 (crack from normal stress in longitudinal weld attaching flange to web), while details 3 and 4 still govern the fatigue capacity of the longer spans. Using HSS and HFMI, the governing detail is close to similar for S460, while using S690 leads to increased fatigue capacity of the HFMI-treated details. This makes detail 2 the governing one all the way until 30 m span.

Table 4.5: Governing fatigue detail in the cost-optimized design of each case. A green cell with a checkmark marks the detail at which the maximum ESR damage occurs.

Case	D1	D2	D3	D4	D5	D6	D7	D8	D9
AW S355, $L = 10$ m				✓					
AW S355, $L = 20$ m				✓					
AW S355, $L = 30$ m			✓						
AW S355, $L = 40$ m				✓					
HFMI S355, $L = 10$ m		✓							
HFMI S355, $L = 20$ m		✓							
HFMI S355, $L = 30$ m			✓						
HFMI S355, $L = 40$ m			✓						
HFMI S460, $L = 10$ m		✓							
HFMI S460, $L = 20$ m				✓					
HFMI S460, $L = 30$ m			✓						
HFMI S460, $L = 40$ m			✓						
HFMI S690, $L = 10$ m		✓							
HFMI S690, $L = 20$ m		✓							
HFMI S690, $L = 30$ m		✓							
HFMI S690, $L = 40$ m				✓					

4.2.1 As-Welded Condition (S355)

Once the PDM optimization is completed for the traditional AW S355 condition, Figure 4.4 illustrates how the structural utilization varies across different spans. The data shows that FLS is the determining limit state for bridges of all investigated spans in some manner, keeping the utilization ratio at approximately 100%. At 20 m, ULS reaches 100% in some sections along the span, but as it is shown in Figure 4.5a, this is not the case for all sections. This indicates that there is still some potential for HFMI counteracting FLS being limiting at this span length. At 30 m, SLS (deflection) has become more limiting than ULS, but FLS is still the governing limit state for AW condition, which suggests there is at least a potential for utilization of HFMI within the domain of increasing SLS to 100%. At the maximum investigated span length of 40 m, SLS and FLS are equally limiting with ULS slightly lower for optimized design. Even though SLS and FLS are limiting for the AW case, the optimized design never the one using maximum allowed total height ($L/15$). This, in combination with the included cost categories, affects the result of the studied cases towards a cheaper design (see Appendix E).

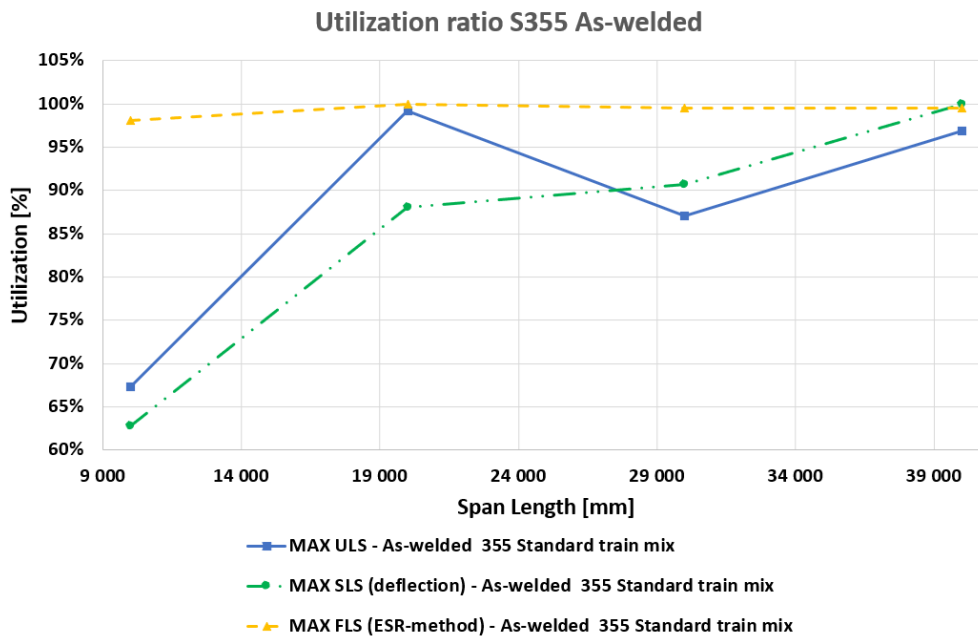
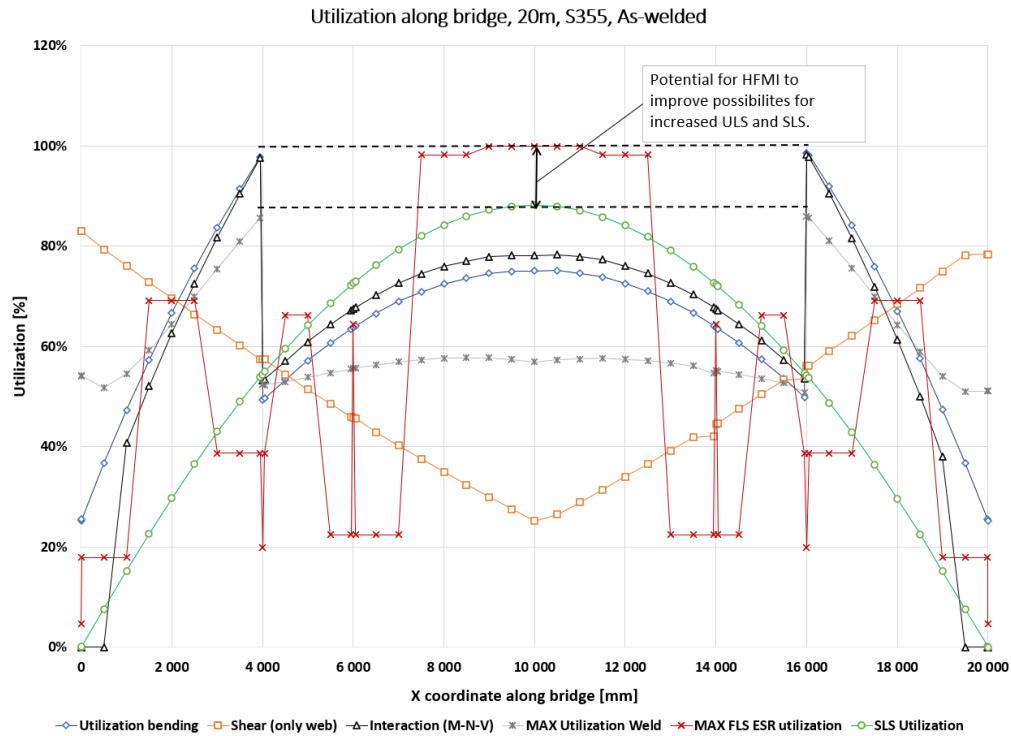


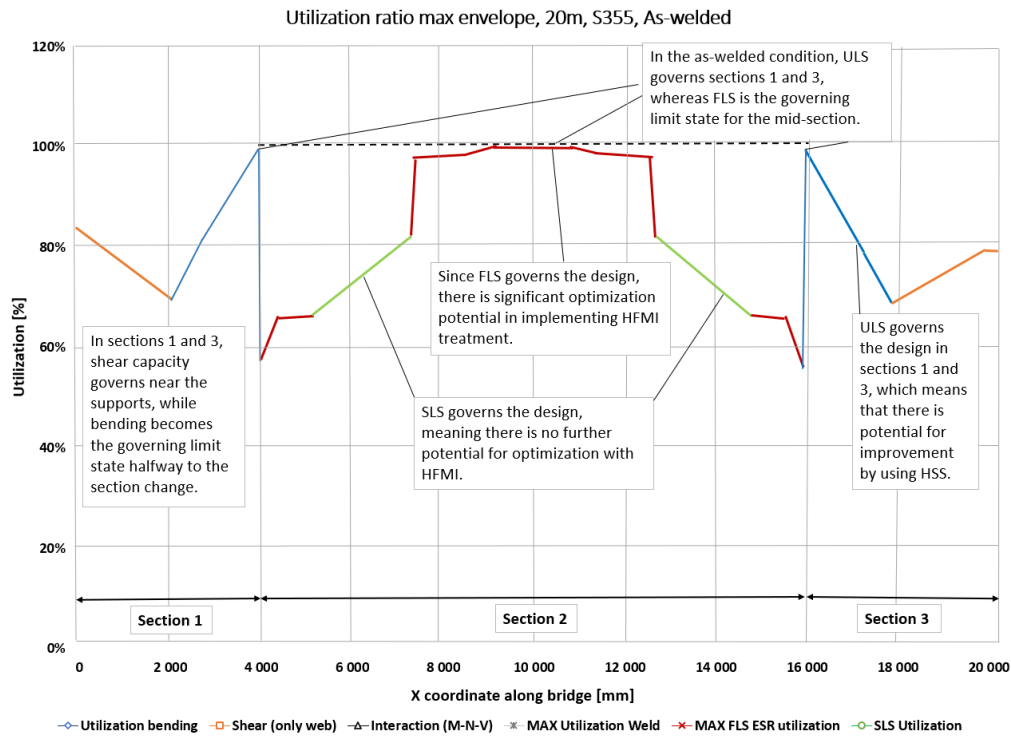
Figure 4.4: Utilization ratios for the AW optimized bridges off 10, 20, 30 and 40 m span lengths.

The behaviour at a span length of 20 m (shown in Figure 4.4) is further studied to understand the behaviour of the model, results, and what limits the design section by section. This case is chosen since it is within the span length where indications show that HFMI results in cost savings, and it is the first bridge length where the curves for both FLS and ULS meet at their maximum utilization ratio in Figure 4.3. This is done by extracting all limit state values along the 20 m optimized bridge. Figure 4.5 shows all limit states section by section, which further explains that both FLS and ULS reach almost a maximum peak of 100% utilization already at 20 m in

Figure 4.4, while there is still some potential for HFMI to improve design.



(a) All utilizations along the AW bridge of 20 m.



(b) Max envelope utilization ratio for the AW bridge of 20 m.

Figure 4.5: Specify which limit is governing for which state (S355 AW).

As shown by the maximum envelope along the 20 m bridge (Figure 4.5), ULS governs the design in sections 1 and 3, while FLS still governs the design in the mid-section. This confirms that there is still some possibilities to improve utilization using HFMI to improve both ULS and SLS. This implies a clear potential for benefiting from HFMI, as shown in Sections 4.2.2 and 4.2.3.

To allow for evaluation of economic effects on the optimized result and which bundles of cost having a greater potential of affecting results distribution between included cost is showcased in Figure 4.6. To further understand the effects of applying HFMI, it is necessary to analyse the cost distribution of the reference case (S355 AW). Figure 4.6 illustrates the total cost distribution for the reference bridges made of S355 in the AW condition across the evaluated spans, with a detailed breakdown of the exact values provided in Table 4.7. As the span increases, the total cost increases exponentially, with materials and painting consistently accounting for the majority of the costs. This cost distribution serves as the primary reference point when assessing the economic profitability of introducing HFMI treatment combined both with S355 and HSS in Sections 4.2.2 and 4.2.3.

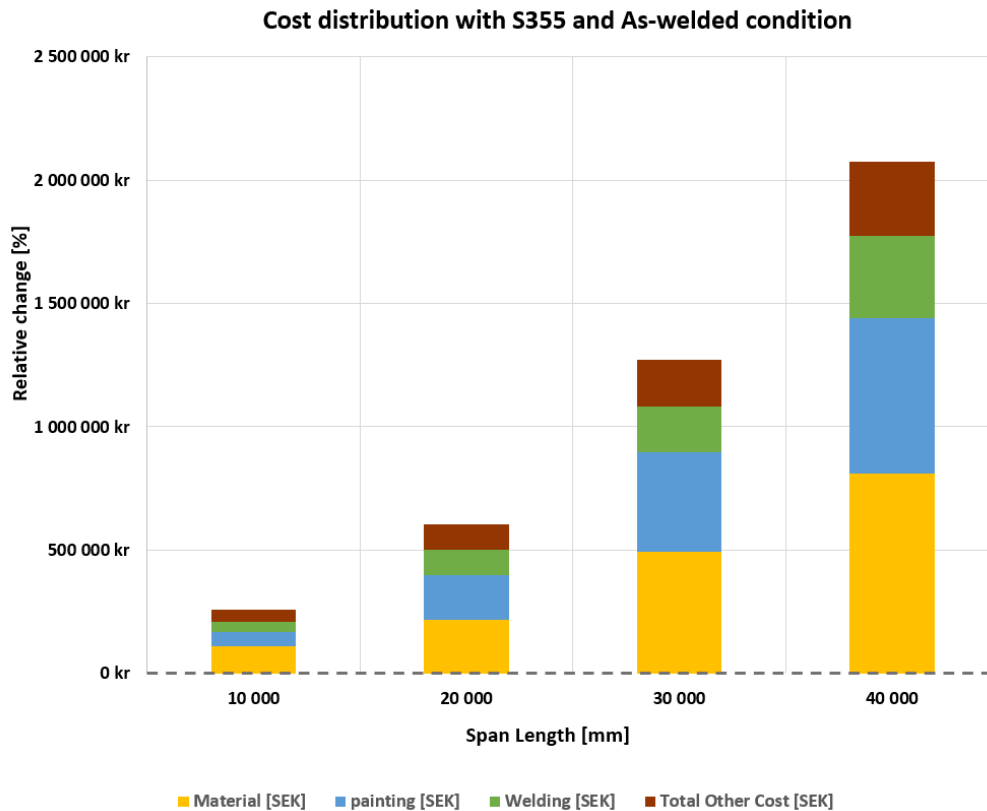


Figure 4.6: Cost distribution for S355 AW for different spans.

4.2.2 HFMI-Treated Condition (S355)

With the reference case evaluated, this section evaluates the effects on limit states and cost through application of HFMI to bridges with steel of grade S355. Figure 4.7 shows the general maximum utilization ratio along all span lengths for the presented case. The most notable effect of HFMI is the significant reduction in FLS, which is also expected. This results in ULS becoming the determining limit state for spans between 10 and 30 m, accompanied by SLS for spans between 20 and 30 m, while SLS is the sole decisive limit at 40 m.

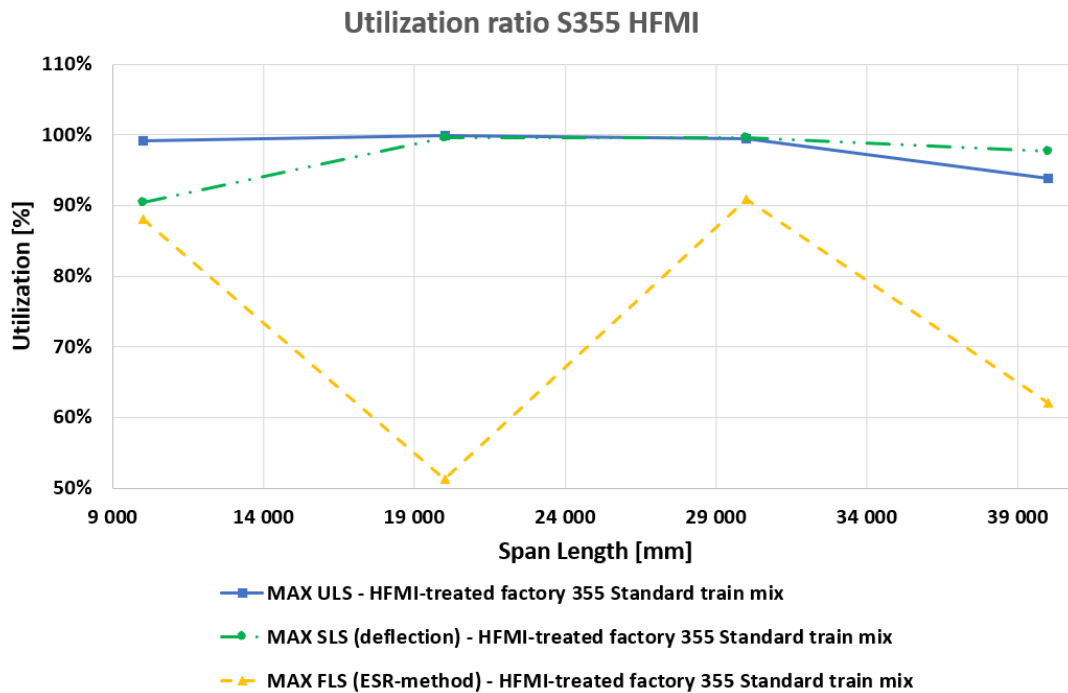
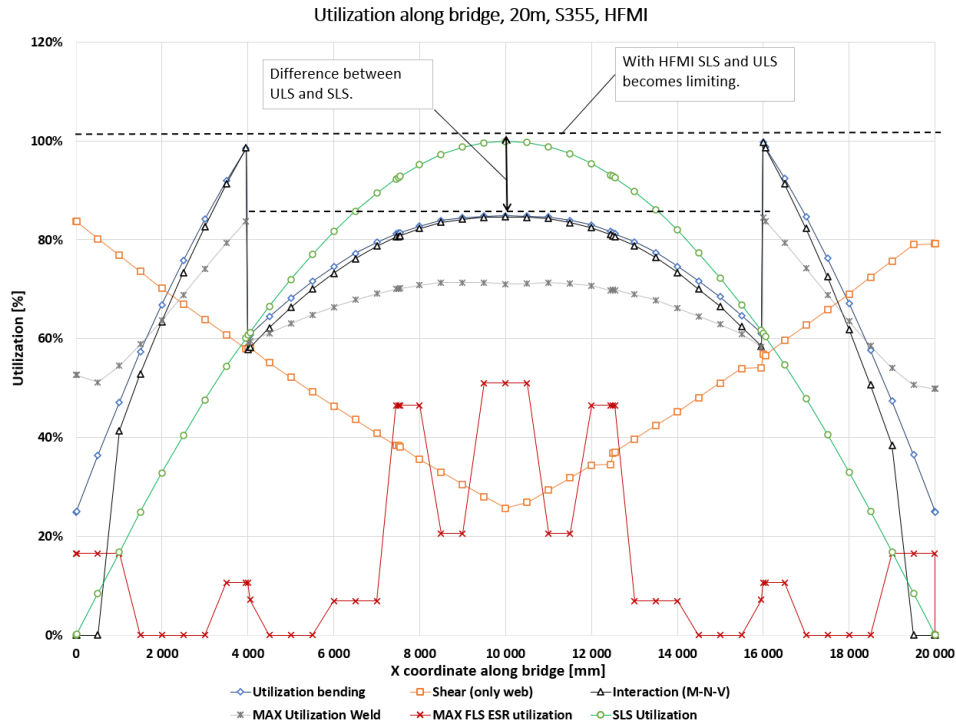


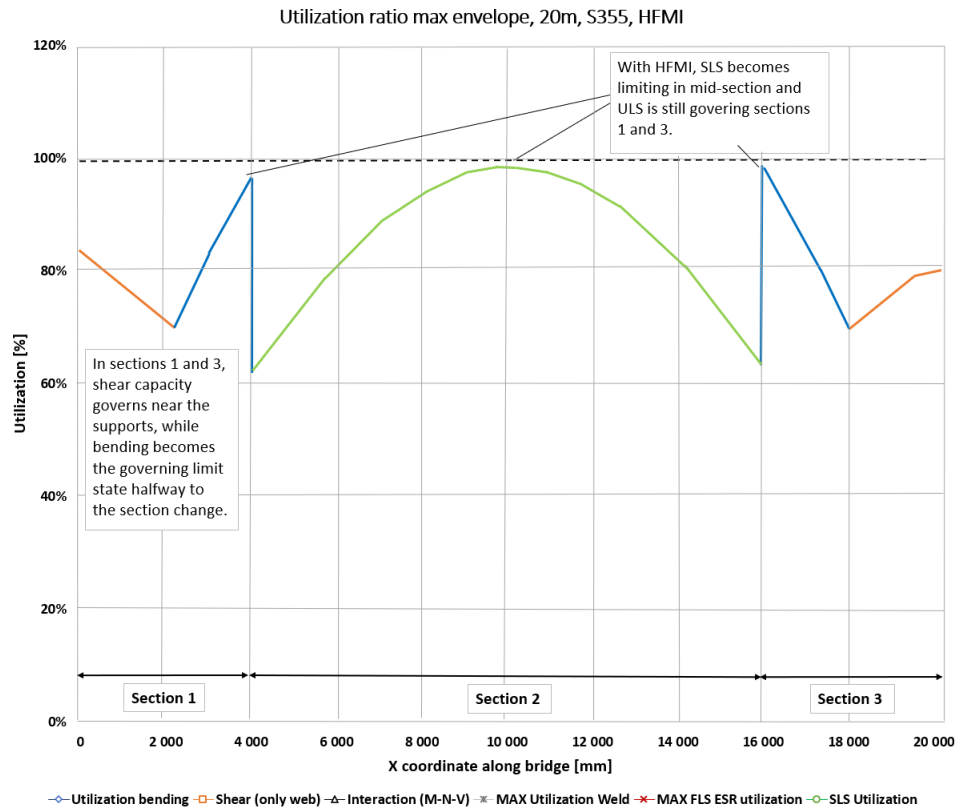
Figure 4.7: Utilization ratios for the HFMI-treated bridges using S355 and for various span lengths.

A similar detailed section-by-section analysis as shown for AW state and 20 m span is illustrated, for the corresponding HFMI-treated case using steel of grade S355, in Figure 4.8. Compared to the AW case in Figure 4.5, FLS is no longer limiting in any of the sections. Unlike the reference case, where the mid-section was determined by FLS, the HFMI-treated bridge has approximately 50% lower fatigue utilization. As shown by the maximum envelope curve (Figure 4.8b), the eliminated limitations from fatigue allow for further cross-section optimization until the other limit states become governing. The support sections (sections 1 and 3) are still governed by ULS (bending), while the middle section is now determined by SLS (deflection), which reaches a utilization ratio of 100%. This shows that HFMI effectively eliminates any limitations from FLS, meaning that the deflection criteria ultimately limit further optimization for S355 spans of this length, in the studied case.

4. Results



(a) All utilizations along the S355 HFMI-treated bridge of 20 m.



(b) Max envelope utilization ratio for the S355 HFMI-treated bridge of 20 m.

Figure 4.8: Specify which limit is governing for which state (S355 HFMI-treated).

As illustrated in Figure 4.9, the application of HFMI enables cross-sectional optimization that directly reduces material usage and total costs. By eliminating the constraints imposed by FLS demands, the cross-section can be optimized until stiffness requirements become the limiting factor. For spans shorter than the point where SLS becomes governing in the AW condition (linearly approximately 34 m), the use of HFMI allows for higher stresses. This eliminates the stiffness requirements induced by AW fatigue limitations, directly resulting in significant material savings. Figure 4.9 shows that for 10 m, the total cost savings are 20%, while the material savings exceed 30%. Spans from 20 to 34 m imply a decreased total cost of 3–5%, with a greater saving in material of 8 to 15%. The greater saving for the 30 m span compared to the 20 m bridge can be traced back to model constraints such as section length limitations and the requirement for symmetric splice welds. In this specific case study, this leads to short support sections of only 4 m for the 20 m bridge, making the welding and manufacturing costs of the splices proportionally more expensive. This combined with ULS already in AW state reaching close to 100% in some sections (see Figure 4.5), while all sections are governed by fatigue at 30 m for the AW state, allows for a greater improvement for a 30 m span using HFMI for the specific studied case. At 40 m, there is no reduction of total cost, but the material cost is decreasing with slightly above 10%, while painting cost increases. This could be linked partially to the earlier presented results of the AW 40 m bridge not utilizing the full possible girder height, when HFMI-treated the material savings in combination with using the maximum possible height result in a cheaper bridge for the specific case. On the contrary while only increasing height for the AW case increases other costs, such as painting more than what is saved in material cost, results in a slightly lower beam. The increased total cost stems partly from the introduced HFMI cost and partly from the stochastic behaviour of the optimization algorithm. Changing the design space to fulfill all limit states shifts the optimization boundaries, resulting in slightly displaced minimum values inherent to such heuristic methods.

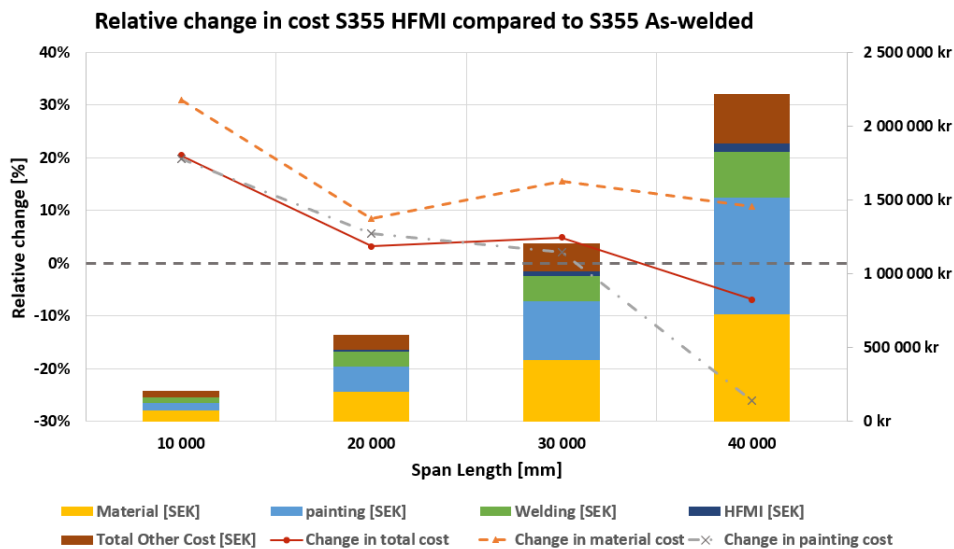


Figure 4.9: Relative change in cost for S355 when HFMI is applied.

4.2.3 High-Strength Steel with HFMI (S460 and S690)

Building on the results from the S355 AW and HFMI-treated condition, this section evaluates whether upgrading to HSS grades in combination with HFMI treatment yields further structural and economic benefits. Figures 4.10 and 4.11 show the limiting utilization ratios across investigated span lengths for S460 and S690, respectively.

In the case of S460 (Figure 4.10), the governing limit state pattern for all span lengths is SLS deflection, while FLS remains well below 100%. However, a key difference is that FLS utilization is consistently lower in this case with S460 than in the case with S355 HFMI, and ranges between approximately 79-88%. This indicates that the increased benefits of HFMI caused by the higher yield strength with S460, provide an additional tolerance margin against fatigue. The ULS utilization ratio is also slightly lower compared to S355 HFMI (except for the 20 m bridge) due to the higher yield strength. Because SLS subsequently becomes the governing limit state, there is no further possibility of reducing the amount of material used in the cross-section.

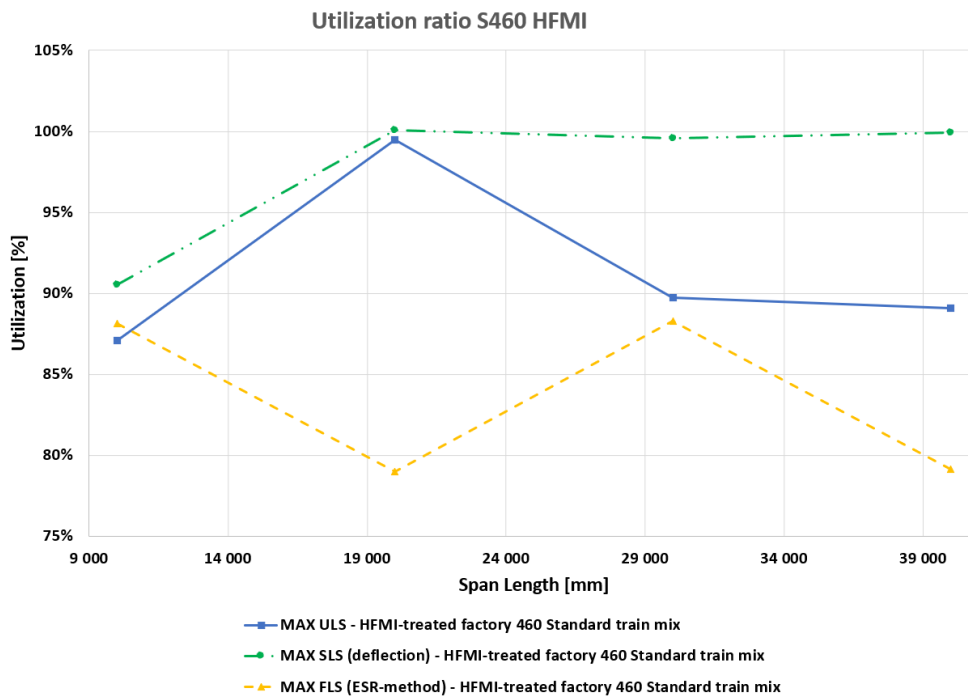


Figure 4.10: Utilization ratios for the HFMI-treated bridges using S460 and for various span lengths.

For the case of S690 (Figure 4.11), a more notable change can be observed. Whilst the ULS utilization begins at around 70% for a 10 m span and gradually rises to approximately 95% at a 40 m span, the FLS utilization at 10 m is similar to S460 since SLS governs both cases. For the longer spans, a distinct reduction in fatigue utilization to around 55% can be observed. This means that for short spans, in this specific case, FLS does not benefit from S690 since AW details already is determinant using S460, but for longer spans, S690 still has an impact on FLS. This

impact and decreased fatigue utilization compared to S460 could be traced to the governing detail becoming an AW detail for spans up to 30 m when using S690, as shown in Table 4.5. With this in mind, the total effect on material usage is still determined by the SLS limit. That means the higher tensile strength and improved fatigue resistance of S690 are not utilized as a consequence of mentioned factors.

The important outcome from these results is that when comparing S460 and S690 the SLS deflection limit, which is independent of yield strength, ultimately constrains both steel grades in the same way at longer spans. This has important implications for the cost analysis in Section 4.3, since the higher material cost of S690 may not be compensated for by savings in the design when SLS governs the design.

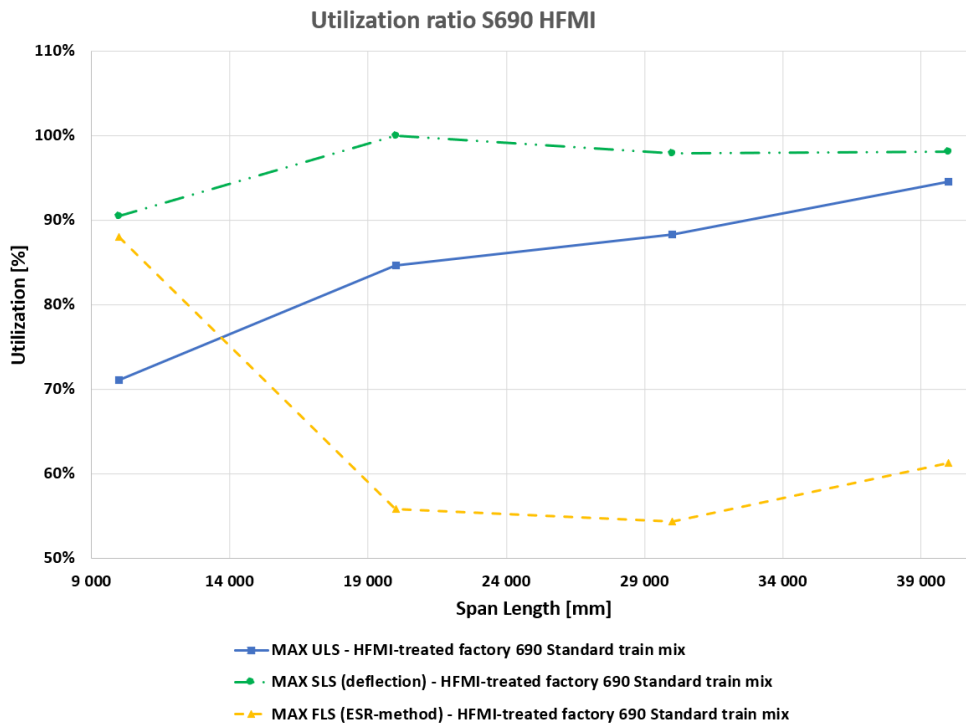
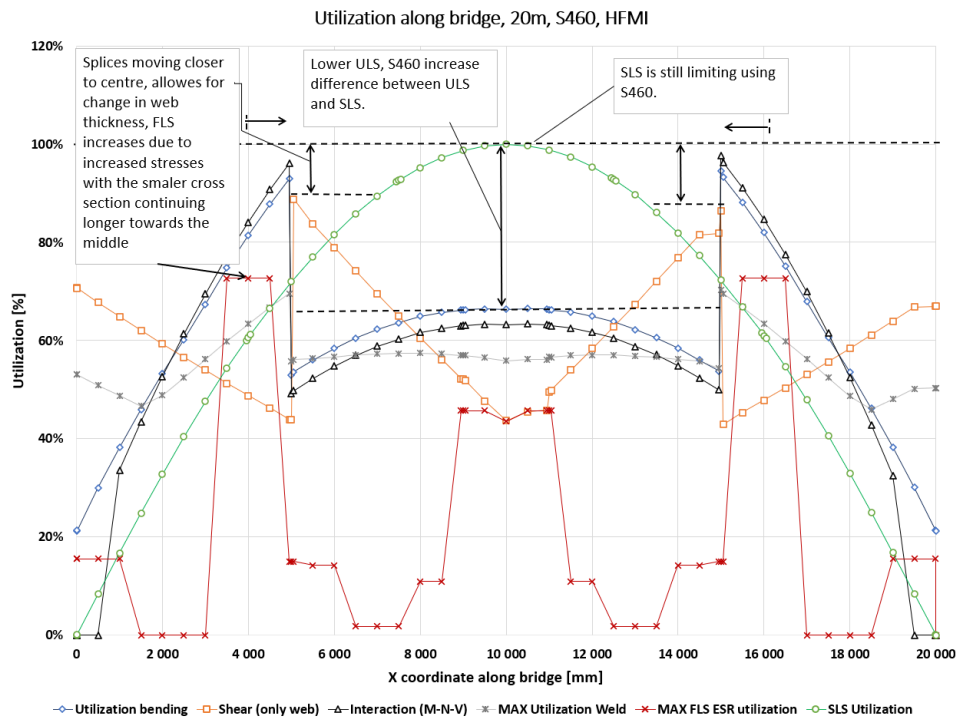


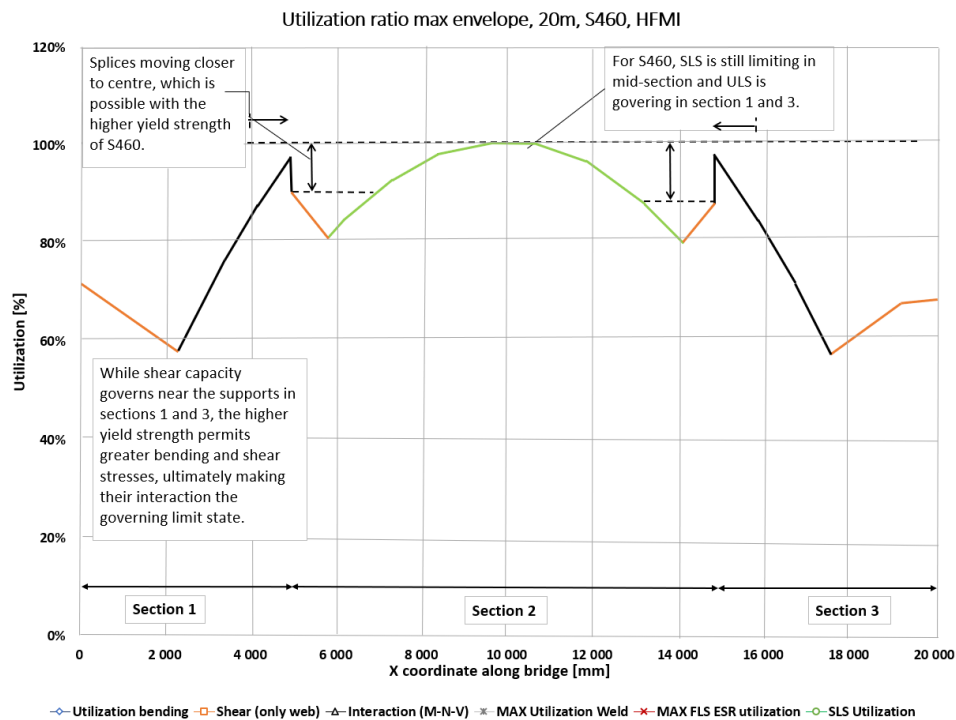
Figure 4.11: Utilization ratios for the HFMI-treated bridges using S690 and for various span lengths.

To examine the local structural behaviour in detail, Figure 4.12 presents the utilization distribution along the 20 m long bridge for the case involving S460 steel treated with HFMI. As shown in Figure 4.12a, the optimized design using S460 moves the joints closer to the middle, without increasing cross-section surpassing SLS limit, though the higher yield strength withstanding higher stresses in ULS. As a secondary effect this also allows the web thickness to be adjusted in the mid-section without interaction being limiting, resulting in higher shear utilization in the mid-section.

4. Results



(a) All utilizations along the S460 HFMI-treated bridge of 20 m.



(b) Max envelope utilization ratio for the S460 HFMI-treated bridge of 20 m.

Figure 4.12: Specify which limit is governing for which state (S460 HFMI-treated).

The maximum utilization ratio curve in Figure 4.12b further clarifies the governing limit states along the 20 m span length. Near the supports (sections 1 and 3), the higher yield strength of the S460 steel allows for greater bending and shear stresses as mentioned. This, in combination with the splices being moved closer to the middle, results in the interaction (M-N-V) becoming the governing limit state, which is a notable change compared to the case with S355 HFMI, where bending was governing in sections 1 and 3. Even though utilization in ULS is similar to the HFMI S355 case, the stresses increase in sections 1 and 3 as mentioned earlier. This leads to an increase in fatigue utilization for detail 3 placed close to the splice in section 1 and 3 (see Figure 4.12) for the specific case (see Table 4.5). In the mid-section SLS remains the governing limit state, similar to the observations for S355 with HFMI configuration. However, the utilization gap between ULS and SLS is notably larger here due to the increased yield strength of the S460 steel.

The changes caused by HSS described above affect the economic efficiency, while some limitations make the usage of the higher yield strength non-profitable. Figures 4.13 and 4.14 show the relative change in cost for the optimal HFMI-treated S460 and S690 designs, respectively, compared with the reference value for S355 and the AW case. The use of HSS combined with HFMI treatment results in cost savings, compared to AW, only for the shortest evaluated span (10 m), where S460 reduces total cost by approximately 13%. However, Figure 4.13 indicates that for spans of, linearly approximated, 17 m and longer, the reduction in material from HSS is not enough to counteract the increased cost, leading to a net increase in costs for S460, both in total and material cost. Similarly, Figure 4.14 demonstrates that for the S690 grade, the increase in cost begins for even shorter spans, resulting in a net cost increase for spans exceeding, linearly approximated, 13 m. This negative economic effect is a direct consequence of the limit states affected by HSS no longer being determinant (see Figure 4.12). Since SLS (deflection) strictly governs the design for these longer spans, the cross-section cannot be reduced further without exceeding the deflection limits. Consequently, the higher material cost of HSS is incurred without a great enough reduction in material usage, restricting the potential cost benefits of material reduction from the increased yield strength.

4. Results

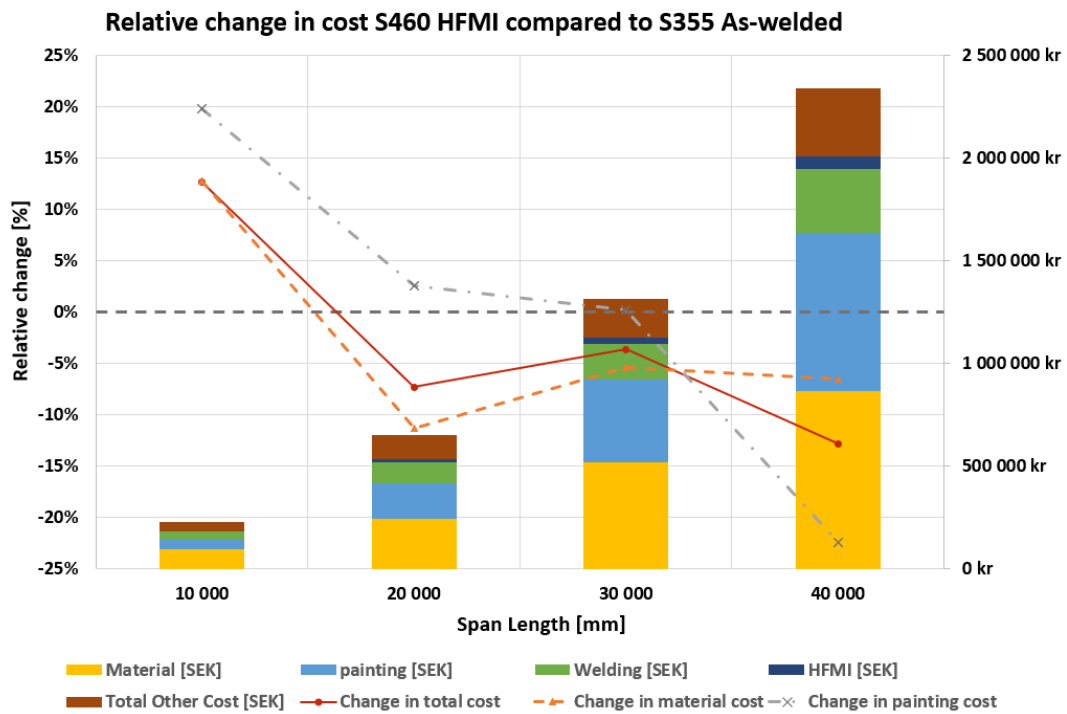


Figure 4.13: Relative change in cost for S460 when HFMI is applied.

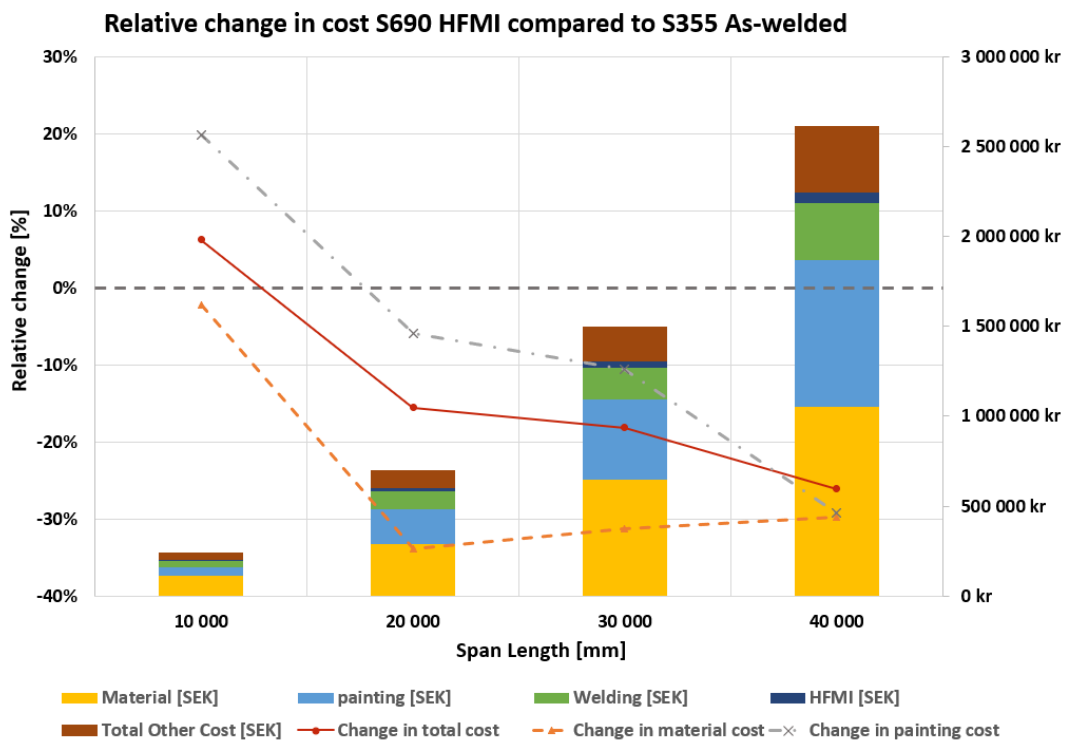


Figure 4.14: Relative change in cost for S690 when HFMI is applied.

4.3 Cost Savings

The PDM optimizes the bridge geometry with a primary focus on production cost-reduction, meaning that full life cycle cost (LCC) is not considered. By utilizing HSS, the dimensions of the structure can be reduced, as earlier stated, when SLS is not governing. This reduction decreases both the total material usage and the surface area requiring surface treatment, which account for the major cost factors (according to the analysis in Section 4.3.1). On the other hand, the economic benefit of using less steel must be weighted against the higher unit price associated with HSS (see Table 3.6). Table 4.6 illustrates this economic trade-off by presenting both the total steel savings and the total cost reduction of HFMI-treated cases, with a conventional AW S355 reference. The data highlights the relative cost difference and clearly demonstrates which combination results in the most economic benefits for the individual span lengths.

Table 4.6: Optimal design configurations, weight savings, and costs for the evaluated steel grades.

Design	Steel Usage [TON]	Weight Saving [%]	Total Cost [MSEK]	Cost Difference [%]
<i>Span Length: 10 m</i>				
Original (AW S355)	9.684	-	0.258	-
Optimized (HFMI S355)	6.638	31.46 %	0.205	20.46 %
Optimized (HFMI S460)	6.638	31.46 %	0.225	12.63 %
Optimized (HFMI S690)	6.612	31.73 %	0.242	6.21 %
<i>Span Length: 20 m</i>				
Original (AW S355)	19.180	-	0.606	-
Optimized (HFMI S355)	17.548	8.51 %	0.587	3.20 %
Optimized (HFMI S460)	16.804	12.38 %	0.650	-7.30 %
Optimized (HFMI S690)	17.326	9.66 %	0.700	-15.58 %
<i>Span Length: 30 m</i>				
Original (AW S355)	43.874	-	1.270	-
Optimized (HFMI S355)	37.122	15.39 %	1.208	4.87 %
Optimized (HFMI S460)	36.944	15.80 %	1.316	-3.64 %
Optimized (HFMI S690)	39.121	10.83 %	1.501	-18.16 %
<i>Span Length: 40 m</i>				
Original (AW S355)	73.292	-	2.075	-
Optimized (HFMI S355)	65.714	10.34 %	2.218	-6.91 %
Optimized (HFMI S460)	61.900	15.54 %	2.341	-12.83 %
Optimized (HFMI S690)	63.893	12.82 %	2.616	-26.09 %

For the 10 m span, all three HFMI-treated configurations result in cost savings compared with the reference case using S355 steel and in AW condition, as shown in Figure 4.15. The greatest saving is achieved with HFMI S355, which reduces the total cost by approximately 20.5%, followed by S460 with 12.6% and S690 with 6.2% (Table 4.6). Regarding material savings, which is larger than the savings of total cost, the three grades achieve almost identical values of approximately 31.5%. The higher unit material cost for the HSS, in combination with this, makes the material cost saving linearly decrease with the size of increased unit price. This implies that

4. Results

HFMI combined with S355 generates the most cost efficient beam at a 10 m span.

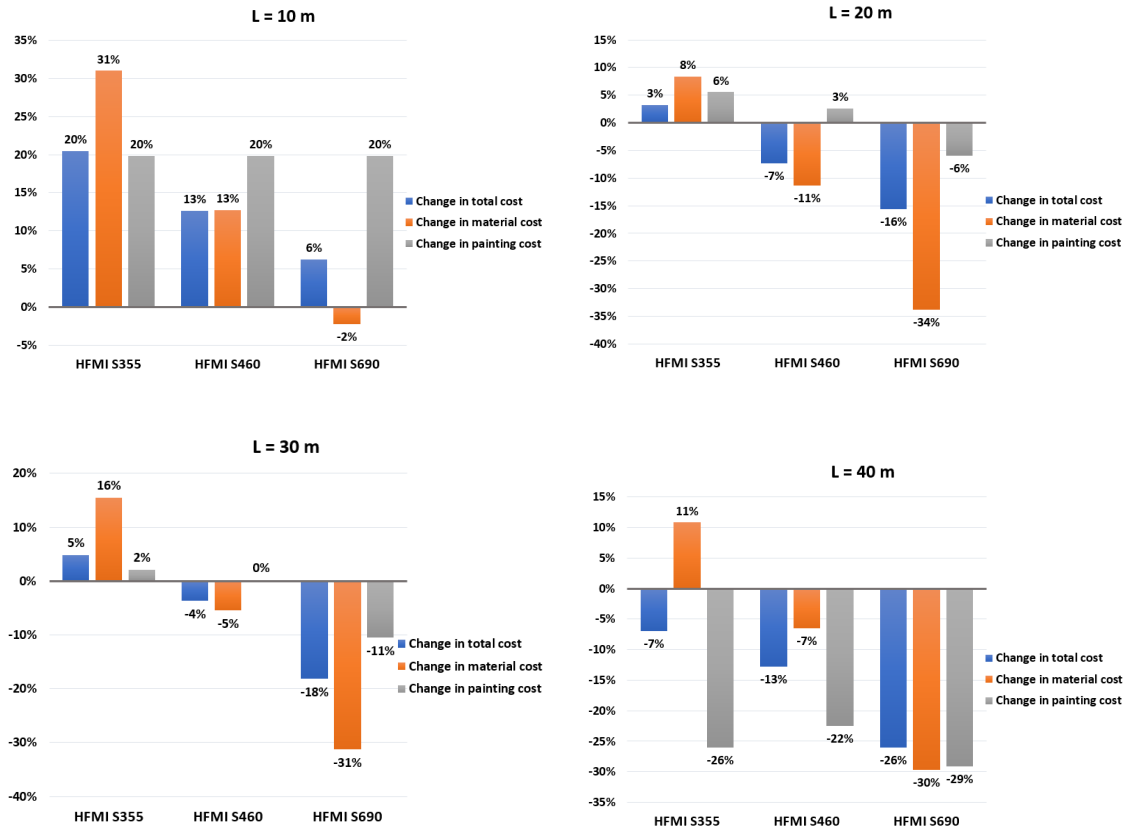


Figure 4.15: Cost ratio of bridges (multiple span length) with HFMI-treated weldments relative to bridge with only as-welded weldments (S355).

At 20 m (Figure 4.15), HFMI S355 delivers a total cost saving of approximately 3.2%, while S460 and S690 result in a net cost increase of 7.3% and 15.6%, respectively. The material savings for the optimized bridge spanning 20 m are lower than for the 10 m span, ranging between 8.5% for S355, to 12.4% for S460 and 9.7% for S690, as shown in Table 4.6. The reason for the large difference in percentage savings is presented in Section 4.2.2. This increased weight reduction of HSS is too small to compensate for the higher steel price, and therefore results in the HSS grades becoming more expensive. At 30 m (Figure 4.15), HFMI S355 provides a slightly larger saving than at 20 m, of approximately 4.9%. Similarly, 20 m S460 and S690 result in cost increases of 3.6% and 18.2% respectively. It is worth mentioning that the weight savings are more notable at this range, particularly for S355 and S460, which achieve reductions of more than 15% according to Table 4.6. However, this further illustrates that the cost disadvantage of HSS is mainly due to the higher unit price in combination with SLS already being limited by HFMI and S355, which results in no further weight reduction from S460, leading to a net economic loss.

At 40 m (Figure 4.15), none of the HFMI-treated bridges offers any cost savings compared with the reference case. HFMI S355 results in an increased total cost of 6.9%, whilst S460 and S690 increase costs by 12.8% and 26.1% respectively. Although weight savings are achieved for each steel grade even at a span of 40 m, with 15.54% for S460 and 12.82% for S690, the higher unit material cost means that no net savings are achieved. This is because the SLS deflection limit restricts further cross-sectional reduction, preventing the full yield capacity of the HSS from being utilized. Regarding the case HFMI and S355, the decreased material cost is lower than the increased surface treatment cost together with the added expense of HFMI, which ultimately makes the bridge slightly more expensive than the corresponding AW bridge.

Although HFMI treatment may not always result in the lowest total cost, it consistently reduces material consumption across all span lengths. Consequently, this reduction has a positive impact when evaluating total steel usage and environmental impact of the steel from cradle to gate.

4.3.1 Cost Breakdown and Distribution

The results shown by Figure 4.16, and precisely defined in Table 4.7, clearly show that materials and painting are the two largest cost categories, consistently accounting for around 60–70% of the total cost for all configurations. Therefore, a key objective of the optimization process has been to reduce these two variables, which has resulted in thicker, less wide plates for the optimized design, which could be seen in Appendix E. Material saving is being done through the width reduction of mainly flanges, while also leading to decreased surface area. Reducing the area that needs to be painted as well as minimizing steel volume through both economic and environmental benefits are achieved, which are discussed further in Section 4.3.2.

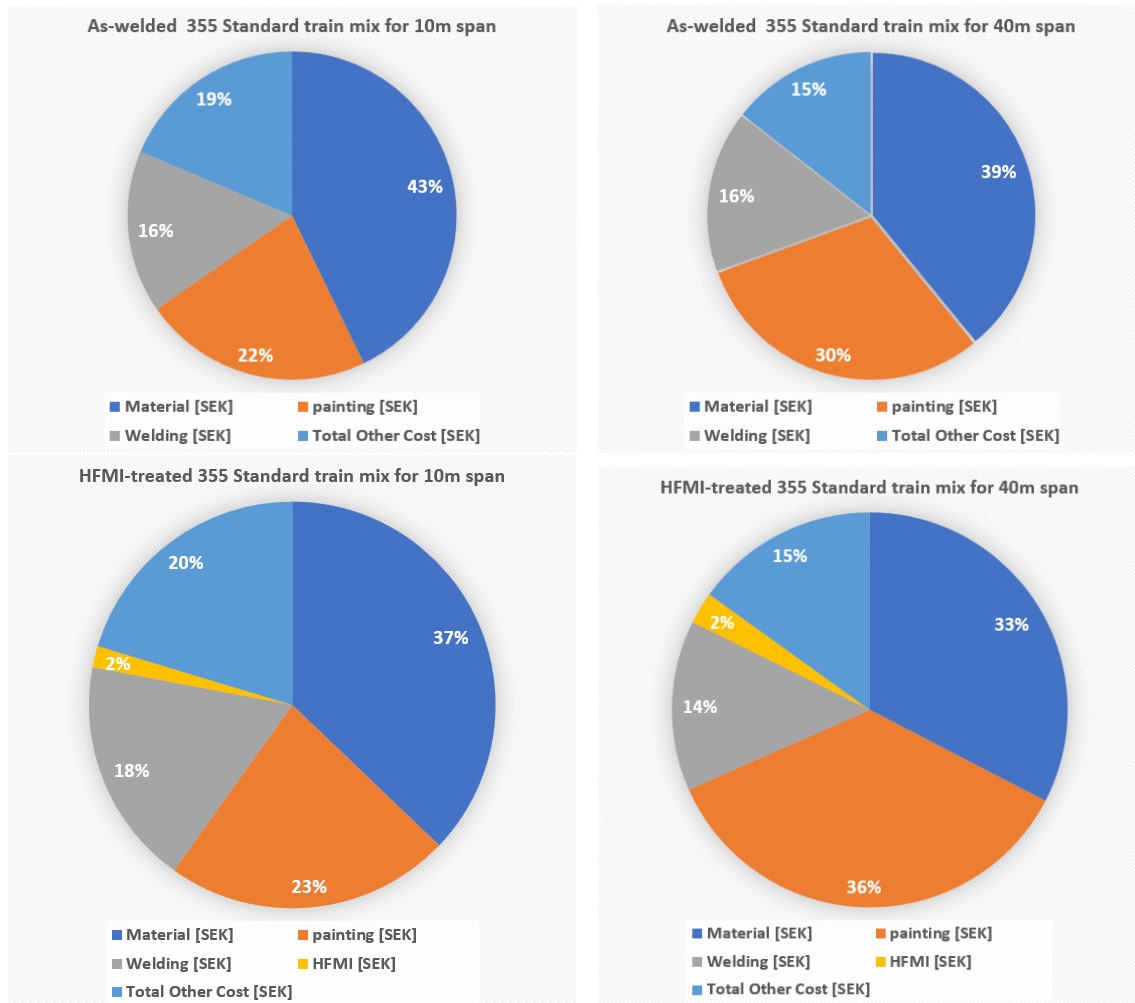


Figure 4.16: Distribution of considered cost for both AW and HFMI-treated bridges spanning 10 and 40m using S355.

A notable trend in Table 4.7 is that the proportion of material costs increases with steel grade. This reflects the higher unit price of HSS despite the reduced steel volume. On the other hand, the paint costs decrease with higher steel grades, which is consistent with the smaller cross-sectional areas achieved. The proportion of weld-

ing costs remains relatively stable for all span lengths and varies between 12–18%. Another trend is change in cost distribution due to changes in span length, visualised both for AW and HFMI-treated case in Figure 4.16. The portion of total cost being material and painting cost increases for longer spans, while some reallocation within the two categories increases the share of painting cost for longer bridges.

Another interesting point is that the HFMI process itself accounts for only 1.5-3% of the total cost in all evaluated cases. This makes the usage of HFMI mainly a question of the possibilities of affecting utilizations, not whether savings are large enough or not to compensate for costs of HFMI treatment.

Table 4.7: Cost share per component, expressed as percentage of the total cost for each case. Rows sum to 100 % within rounding.

Span	Case	Material	Painting	Welding	HFMI	Other
10 m	AW S355	42.8%	22.5%	16.0%	0.0%	18.6%
	HFMI S355	37.1%	22.7%	18.1%	1.7%	20.3%
	HFMI S460	42.8%	20.7%	16.5%	1.5%	18.5%
	HFMI S690	46.7%	19.3%	15.4%	1.4%	17.2%
20 m	AW S355	36.0%	29.6%	17.4%	0.0%	17.0%
	HFMI S355	34.0%	28.9%	17.2%	2.8%	17.1%
	HFMI S460	37.3%	26.9%	15.4%	2.5%	17.9%
	HFMI S690	41.6%	27.1%	14.6%	2.5%	14.2%
30 m	AW S355	38.7%	31.9%	14.6%	0.0%	14.7%
	HFMI S355	34.4%	32.8%	14.3%	2.6%	15.8%
	HFMI S460	39.4%	30.7%	13.1%	2.4%	14.4%
	HFMI S690	43.0%	29.8%	11.9%	2.2%	13.0%
40 m	AW S355	39.1%	30.3%	16.1%	0.0%	14.5%
	HFMI S355	32.6%	35.8%	14.0%	2.6%	15.1%
	HFMI S460	36.9%	32.9%	13.4%	2.5%	14.3%
	HFMI S690	40.2%	31.1%	12.3%	2.2%	14.1%

4.3.2 Reduction of Material Usage and CO_2 Emissions

As mentioned previously, in this thesis, the optimization is done with respect to cost, not the CO_2 -emissions. However, in the following section, impact on CO_2 -emissions will be examined. The reduction of emission is achieved through decreasing the required amount of material and possibly also by reducing the painting area. Table 4.8 presents effects related to the material usage, through total amount of material savings achieved, in the optimization of different cases investigated. Material savings, both related to usage of HFMI treatment and effects from higher steel grades, are presented. To further compare a possible decreased environmental impact using HSS, comparably, CO_2 -equivalents (from material only) of optimized designs (based on values in Figure 3.21) are also included in Table 4.8. As Table 4.8 shows, HFMI-treated S690 is the least favourable for any case compared to results from using other steel grades, as its total CO_2 emissions remain very similar to those of the original S355 structures (generates only a 0.40% reduction at 20 m). In comparison, it can be seen that HFMI-treated S355 and S460 consistently result in better environmental performance than the AW case.

Figure 4.17 further illustrates these trends by comparing the relative reduction in both material usage and carbon dioxide emissions for different steel grades and spans. A consistent pattern is observed for all spans, with weight savings and carbon dioxide reductions being similar for S355 and S460, with a significant difference between the values for S690. This reflects the higher carbon dioxide emission factor associated with the production of S690 (1.72 Ton $CO_{2,eqv}$ /Ton steel compared to 1.56 for S355), which slightly reduces the environmental benefit of any reduced steel volume. The additional material saving for S460 at the span of 20 m could be explained by Figure 4.18, presenting the relative change in cross-section area of the individual plates in the main girder. Comparing the reduction of bottom flange area S355 and HFMI offers a reduced area throughout all sections, while S460 has a larger reduction in support sections and a slight increase in the mid section. The top flange has a slightly decreased cross-section in all sections comparing S460 HFMI and S355 HFMI. The main cause of decreased material, presented in Figure 4.17, using S460 is though a decreased web cross-section in the mid span using S460 (see Figure 4.18b).

Figure 4.17 shows that at 10 m, the three steel grades achieve significant reductions in CO_2 emissions compared with the AW reference case. The S355 and S460 reduce emissions by 31.5% and 28.8% respectively, and the S690 achieves a reduction of 24.7%. At 20 m, this difference becomes more noticeable. S355 and S460 are still showing similar reductions of 8.5% and 9.0% respectively, but S690 drops to just 0.4% reduction. At 30 m, S355 and S460 still show significant reductions of 15.4% and 12.6% respectively, while S690 only achieves 1.7%. At 40 m, S460 delivers the greatest CO_2 reduction, with 12.3%, which is slightly better than S355, with 10.3%, while S690 once again results in the least improvement, with only 3.9% reduction. As for the increased saving of S460 at 40 m could be explained by a similar behaviour as shown explicitly at 20 m in Figure 4.18b, which also explains the weight gap between HFMI S355 and HFMI S460 at 40 m.

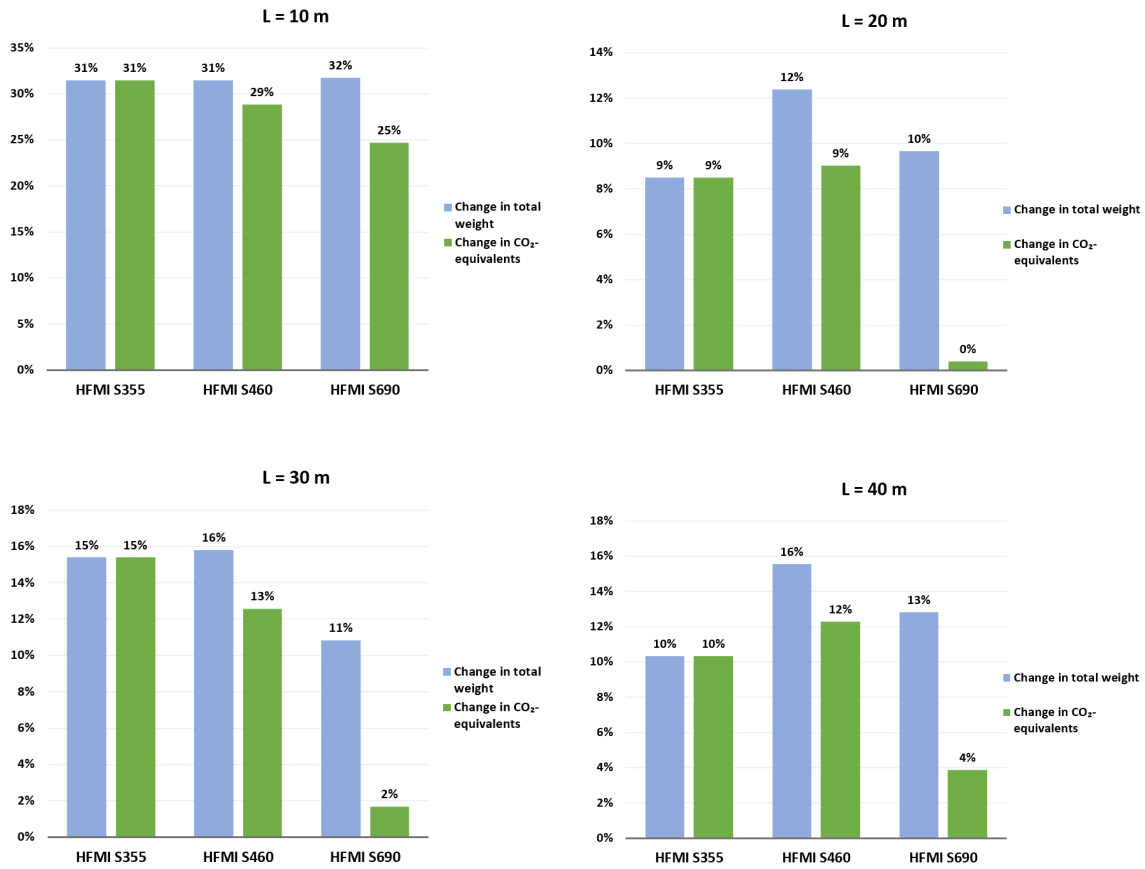
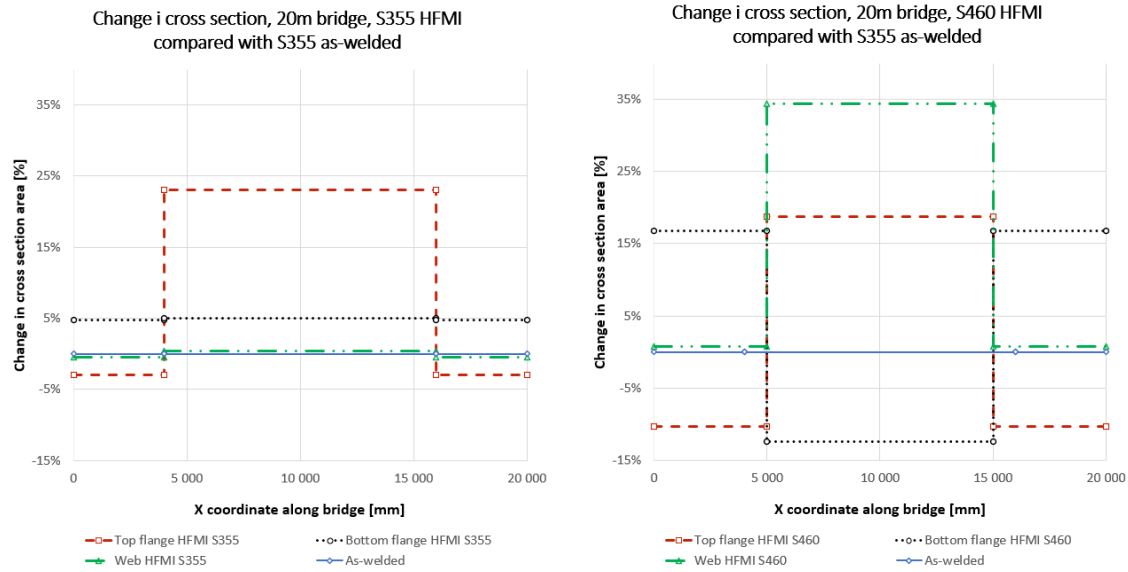


Figure 4.17: Change in material usage and CO_2 -emissions for bridges spanning 10, 20, 30 and 40 m, with a reduction compared to AW state being defined as a positive percentage change.

The reason why S690 consistently achieves least reduction in emissions, despite achieving comparable or slightly greater weight savings, is its increased carbon footprint per weight unit. For S690, the reduction in steel volume is not enough to make up for the higher emission factor, in any of the spans, leading to total CO_2 values approaching the reference value for AW S355, presented in Table 4.8. With this in mind, if steel industry in the future are able to produce HSS with a lower carbon foot print these results will drastically change.

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(a) Change in plate cross-section area, at a span of 20 m using HFMI and S355. (b) Change in plate cross-section area, at a span of 20 m using HFMI and S460.

Figure 4.18: Relative change in plate cross-section area, at 20 m span, comparing AW S355 with HFMI S355 and S460

Overall, the results show that HFMI treatment consistently reduces both material usage and CO_2 emissions in all the evaluations. The HFMI-treated steel grades S355 and S460 deliver the most beneficial environmental outcomes, meanwhile S690 with current emission values does not gain enough savings from its increased yield strength, particularly for longer spans where deflection is limiting already when using lower grades such as S355.

Table 4.8: Material usage and estimated CO_2 emissions based on total weight for different span lengths.

Design	Steel Grade	CO_2 per Ton*	Total CO_2 [Ton]	Change in CO_2
Span Length: 10 m				
Original (AW)	S355	1.56	15.107	-
Optimized (HFMI)	S355	1.56	10.355	31.46%
Optimized (HFMI)	S460	1.62	10.754	28.82%
Optimized (HFMI)	S690	1.72	11.373	24.73%
Span Length: 20 m				
Original (AW)	S355	1.56	29.601	-
Optimized (HFMI)	S355	1.56	27.375	8.51%
Optimized (HFMI)	S460	1.62	27.222	9.01%
Optimized (HFMI)	S690	1.72	29.801	0.40%
Span Length: 30 m				

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Table 4.8 – Continued from the previous page

Design	Steel Grade	CO ₂ per Ton*	Total CO ₂ [Ton]	Change in CO ₂
Original (AW)	S355	1.56	68.443	-
Optimized (HFMI)	S355	1.56	57.910	15.39%
Optimized (HFMI)	S460	1.62	59.849	12.56%
Optimized (HFMI)	S690	1.72	67.288	1.69%
<i>Span Length: 40 m</i>				
Original (AW)	S355	1.56	114.336	-
Optimized (HFMI)	S355	1.56	102.514	10.34%
Optimized (HFMI)	S460	1.62	100.278	12.30%
Optimized (HFMI)	S690	1.72	109.896	3.88%

*Emission factor for structural steel, based of Figure 3.21 [Ton CO_{2,eqv}/Ton steel].

5

Discussion

This thesis has aimed to investigate the scale of, in which cases and using certain configurations HFMI treatment could affect production cost and material usage. To get a realistic result, this has been studied through a case study using a simply supported, single track, double I-girder bridge design as a basis. The subject is then investigated through a developed PDM used in combination with GA optimization to find the optimal solution, with respect to lowest investment cost, for the different studied cases of the reference bridge. To then further analyse what the findings implies about possibilities and effects offered by HFMI treatment.

The implementation of HFMI treatment on the investigated welded details significantly reduces the fatigue utilization. In the reference AW cases, fatigue acts as the governing limit state across all investigated spans. However, the enhanced fatigue capacity provided by HFMI shifts the governing limit state toward ULS for shorter bridges and SLS for spans over 20 m. This shift enables a more material efficient geometry, leading to direct reductions in steel consumption and overall costs for the S355 designs up to linearly approximated 34 m. While HSS (S460 and S690) offer higher yield strength, their enhanced potential of cross-section optimization is ultimately restricted by the yield-independent deflection criteria in SLS. This effectively limits their potential economic and environmental benefits for longer spans, only showing a cost improvement of S460 at a 10 m span.

Since the optimization has been conducted, including several categories of cost, some particularly interesting findings have been discovered related to those, both regarding the AW case and changes from the implementation of HFMI. One of them being the large portion of the total cost consisting of painting costs for all studied cases, ranging between 20-36%. This significant painting cost also offers a potential for savings, which has proven to be the case, especially for shorter bridges. On the contrary, for longer bridges restricted by deflection in combination with fatigue the observations from the results indicate an increased painting cost but decreased material cost. For shorter bridges (up to 30 m), the 2% to 20% reduction in painting costs achieved through HFMI treatment could have an even greater impact if considering a full life-cycle evaluation of cost and environmental effects. This indication could be strengthened by the results from similar studies conducting optimization of steel railway bridges, such as the one by Hlal, 2025. Hlal's study showed that when full LCC are evaluated, painting can account for as much as 36% to 55% of the total cost. This implies that an optimization with a broader objective function, including life cycle effect such as this one, could have influenced optimized designs

to be even more influenced by reduction of painting area.

To ensure the reliability of the economic analysis, unit prices for structural steel have been received from market average at SSAB, while manufacturing and painting costs are based on current industry data used in similar studies (Hlal, 2025). Although these costs are naturally subject to normal market fluctuations, minor price variations would only slightly shift the profitability thresholds between the different steel grades. For longer spans, where SLS deflection restricts further cross-sectional reduction regardless of yield strength, the economic profitability of HSS remains limited regardless of potential price reductions. For shorter spans, where the design is governed by fatigue or ULS, a reduction in HSS material costs could however extend the span range where S460 or S690 become economically competitive.

Although it was not a direct constraint in the optimization algorithm, it is worth highlighting the evaluation of the dynamic properties of the optimized bridges as an interesting side note. A retrospective analysis of the eigenfrequency (n_0) shows that the majority of the optimized structures (for spans of 20, 30 and 40 m) naturally falls well within the permissible limits specified in Eurocode *SS-EN 1991-2: Traffic loads on bridges* (Swedish Standards Institute, 2003), for the assumed velocity. However, an exception was observed for the 10 m bridges, where the optimized geometry resulted in an eigenfrequency exceeding the upper acceptable limit. A very interesting observation for the 10 m span in particular is that the use of HFMI actually reduced the eigenfrequency compared to the untreated AW case, thereby improving the value closer to the acceptable limit. This reduction could be explained by FLS no longer being limited by the required stiffness, leading to a cross-section with slightly larger deflection from self weight decreasing the eigenfrequency. This observation is in line with the results presented by Hofstetter et al., 2026, which demonstrated that for steel railway bridges, eigenfrequency requirements become even more critical as train speeds increase. Although this study was conducted for conventional train speeds, the results emphasise the importance of including eigenfrequency and dynamic behaviour as specific boundary requirements in future studies, particularly if investigating high-speed train scenarios. Especially since an increased train speed could affect the outcome, shift the limiting state, and then affect when and to what extent HFMI treatment with a certain configuration is beneficial in combination with higher train speeds.

The selection of optimization parameters, particularly the geometric boundaries, significantly influences the design outcome. While standard industry practice often limits girder height to between $L/20$ and $L/30$ due to clearance requirements according to Collin et al., 2012, this study permitted a broader range ($L/15$ – $L/40$). Sensitivity analyses detailed in Appendix F demonstrate that restricting the height to $L/20$ increases total costs by 15–30% and material usage by 20–45%. This confirms that for projects without strict clearance limitations, challenging conventional height boundaries is essential to fully utilize the economic and environmental benefits of HFMI and HSS designs.

There are two methods used for fatigue designs, the one mainly used in this study being the ESR method, while another commonly used method being the λ -method also explained in chapter 3. With the λ -method being a calibrated method using a static load and calibrated factor to represent the dynamic effects of moving trains often being slightly more conservative. Investigating the potential of HFMI treatment using this specific method is outside the scope of this thesis, but it could still provide interesting insights. As a sensitivity check of method dependency to benefits from HFMI treatment optimizations using λ -method for spans from 10 to 40 m, for AW and HFMI-treated case using S355 has been done from which results are presented in Figures 5.1 and 5.2, and the preformed Excel calculations can be found in the Appendix G. These results indicated fatigue limit state being slightly more restrictive for the AW case, which is expected from the more conservative method, with FLS being the limiting state also for the 40 m span. As a cause of this, implementing design including HFMI treatment, relative change of cost becomes even greater when using the λ -method (see Figure 5.1). When using HFMI treatment and S355, the governing limit state shifted to deflection for most spans in this case as well. It is important to note that the actual optimized prices proved to be the same for both methods, as a result of reaching the deflection limit when applying HFMI. Which implies that the greater improvement comes from a more conservative and expensive AW design using the λ -method. In summary this indicates that the potential of improvement is method independent, and that the relative change seems to be even greater using the λ -method. An additional observation made is that the difference in AW design between the methods is less at shorter spans while it becomes increasingly greater at longer spans. This could indicate that the added calculation time using a method as the ESR method instead of λ -method is not worth at shorter spans using both AW and HFMI treated design. With this in mind it also implies that the increasingly calculation heavy ESR-method becomes more meaningful at longer spans leading, especially for AW design.

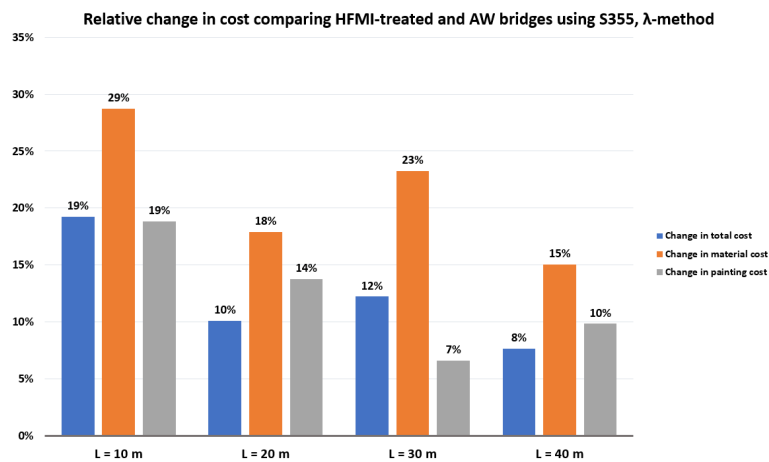


Figure 5.1: Relative change in cost comparing HFMI-treated and AW bridges of different span lengths using S355 and λ -method.

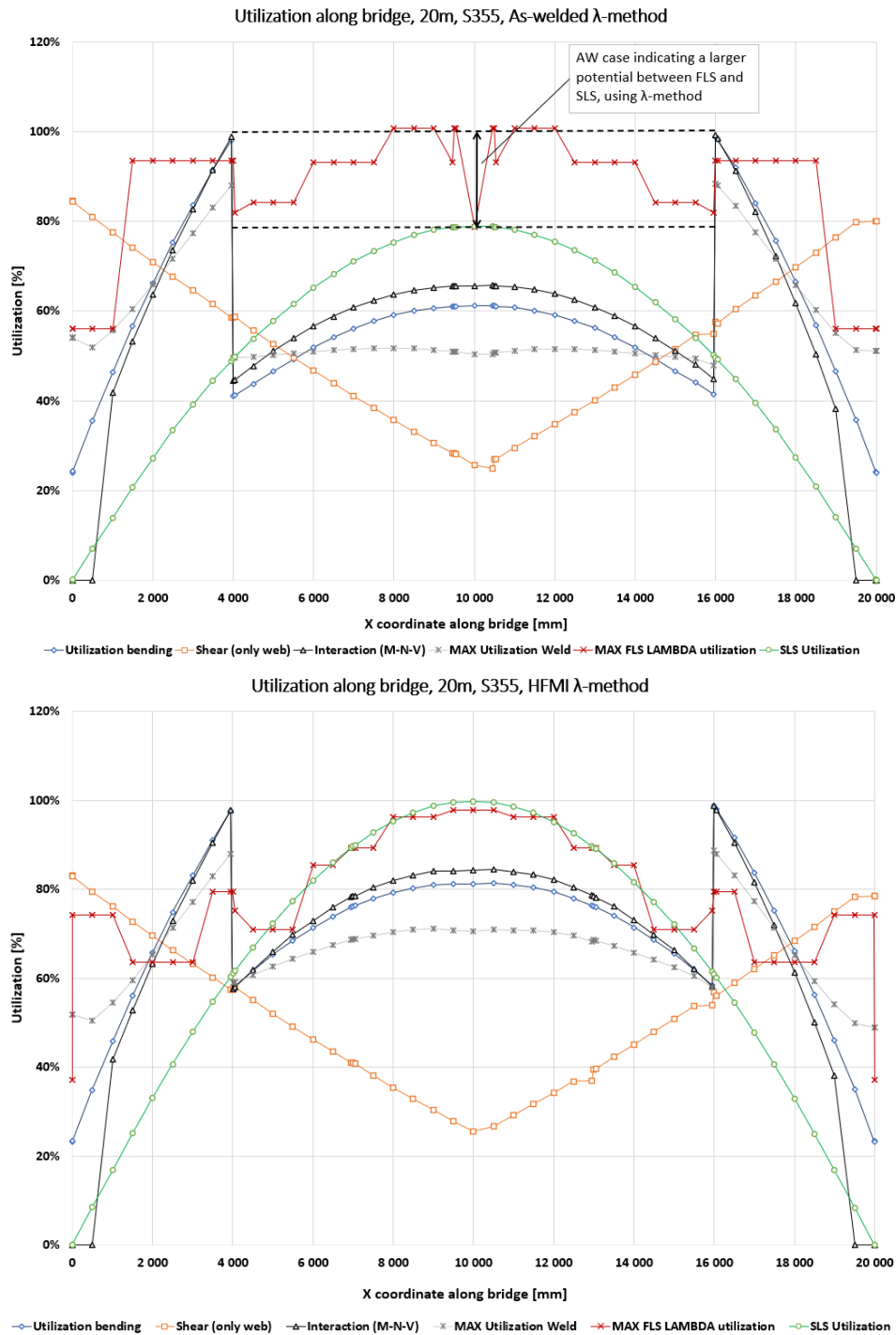


Figure 5.2: Utilization ratios along 20 m bridge, for AW and HFMI-treated case using S355 and λ -method for fatigue verification.

One aspect that repeatedly appears in the results is that, once HFMI treatment is applied and fatigue is no longer the governing limit state, SLS deflection becomes the decisive limiting factor for longer spans, while ULS determines shorter ones. This ultimately restricts how much further optimization can be achieved through the use of HSS in combination with HFMI. Unlike ULS and FLS, where the load-bearing

capacity can be directly increased by increasing the strength of the material's yield or by improving the fatigue strength through PWT, the deflection is affected solely by the stiffness of the cross-section and Young's modulus. Since Young's modulus of steel is independent of steel grade, an increase in the yield strength from S355 to S460 or S690 does not lead to any improvement of deflection. The only way to reduce deformations is to increase the cross-section's moment of inertia, which requires larger geometric dimensions and thus more material, thereby counteracting the weight and cost savings achieved with HFMI and HSS. This limitation is particularly evident for the simply supported bridges evaluated in this study. For this static system, maximum deflection occurs at mid-span, which inevitably becomes the governing criterion in SLS for longer spans. A continuous beam configuration could distribute the bending moment in another way, which could potentially postpone the limitations from deflection, allow for further increase of same total length bridge divided into several spans (as studied by Englund and Dagduk, 2022). In summary, the structural advantages of HFMI cannot be fully utilized for cross-section optimization in spans where SLS deflection is critical. Therefore, the benefits of the treatment are primarily realized in configurations where fatigue or ULS represent the critical design limits.

Previously conducted studies have mainly focused their investigations on steel-concrete composite bridges where the concrete deck continuously restricts the top flange, inherently eliminating the risk of lateral-torsional (LT) buckling (such as the one by Englund and Dagduk, 2022). Therefore, a core question of this thesis was to uncover whether a instability phenomena would become governing for bridges with non-horizontally constrained compressed flanges, such as the open twin I-girder railway steel bridges optimized in this thesis. The results of this verification are presented in Figure 5.3, showcasing the dispersion of buckling factors for the two primary stability modes across all optimized designs in both AW and HFMI-treated conditions. The data clearly indicates that the more slender, optimized cross-sections never face stability issues. For the global combined flexural and lateral-torsional buckling mode, the factors consistently remain well above the critical threshold of 1.0 (with some values below 1.5 for bridges shorter than 30 m). Simultaneously, the local out-of-plane buckling mode between transverse elements is safely maintained below 1.0 (with some utilization ratios closer to 1 at span length 20 m), verifying that the horizontal bracing system provides highly efficient lateral restraint to the compression flanges. In the open double I-girder design, SLS deflection limits always become decisive and dictate the minimum required flange dimensions well before LT-buckling has a financial or physical chance to occur. This demonstrates that while LT-buckling is a theoretically relevant issue for open steel girders with unrestrained compression flanges, a well adapted horizontal stiffening system combined with SLS deflection requirement ensures robust stability margins across all evaluated span lengths without restricting the efficiency of HFMI as a tool for design optimization.

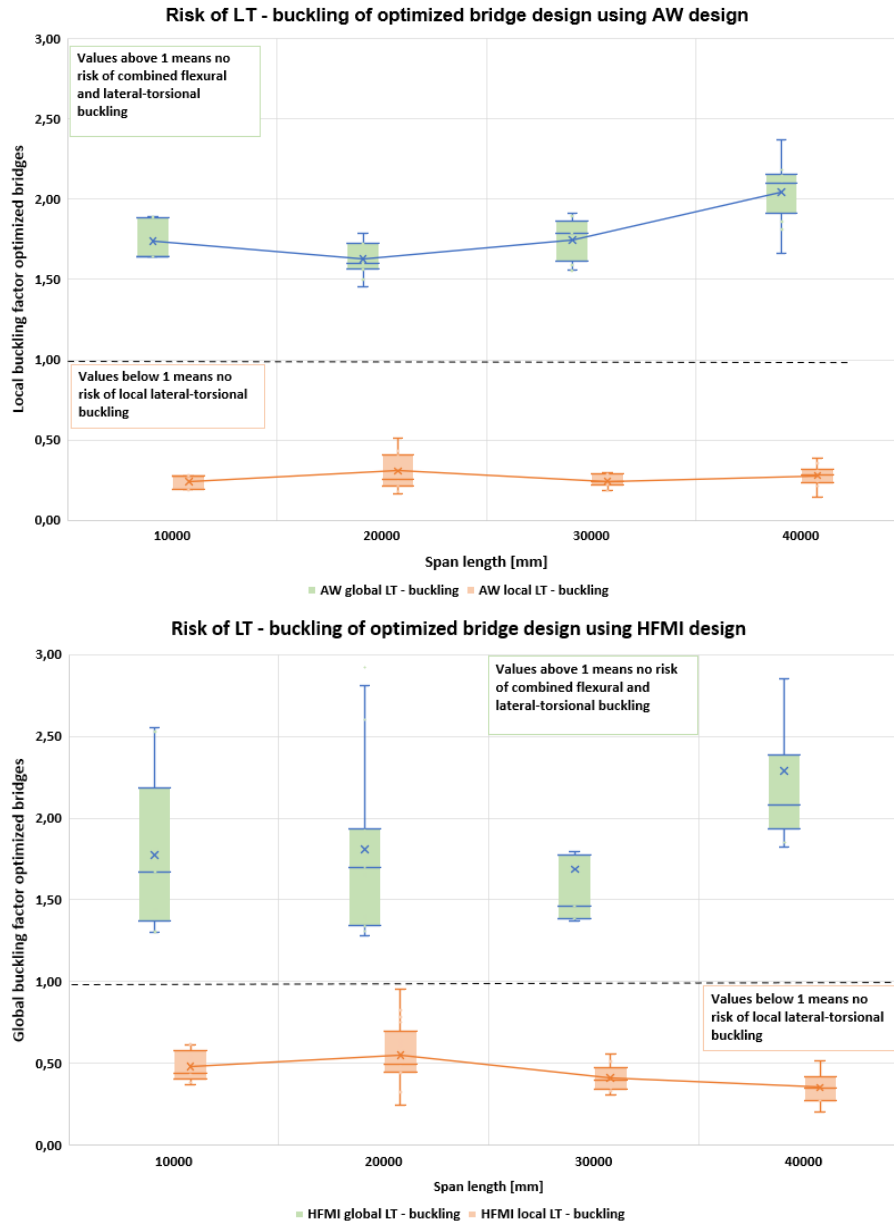


Figure 5.3: Factor against global combined flexural and lateral torsional buckling (above 1), and local lateral torsional buckling (below 1) of AW and HFMI-treated optimized design.

Furthermore, the optimization results indicate that certain parameters could be excluded in future studies, since they not affect the results of the studied subject. These parameters primarily include the throat thickness of the longitudinal weld (assuming costs do not increase with HSS), as well as the sizes of the UPE cross-beams and L-profiles. These elements are not directly governed by the HFMI-driven cross-section optimization, only changing the effective buckling length of the local LT-buckling, which never becomes the decisive design factor of the studied case with the included design criterion (see Appendix D). From the ULS controls considered bending, shear and interaction of moment, normal force and bending is governing shorter spans (until about 30 m) while the 40 m bridges mainly is constrained

by the control of flange induced buckling. The SLS utilizations also shows the deflection to not be an issue of shorter bridges while it becomes the governing one when HFMI is used. Eigenfrequency that was not considered as a constraint in the optimization only surpasses the limit value at the shortest AW bridge, while the correspondence HFMI-treated one not suffers from the same issue. This is an effect of HFMI allowing for reduced sectional stiffness, consequently altering the dynamic response and eliminating the eigenfrequency issue trough slightly increasing deflection. Lastly, fatigue only governs the design of the AW bridges, whereas all HFMI-treated ones have a fatigue utilization of 90% or lower.

Table 5.1: Governing limit state of each investigated case.

Case	ULS	SLS	FLS
AW S355, $L = 10$ m			<i>ESR damage</i>
AW S355, $L = 20$ m			<i>ESR damage</i>
AW S355, $L = 30$ m			<i>ESR damage</i>
AW S355, $L = 40$ m		<i>Deflection</i>	
HFMI S355, $L = 10$ m	<i>Shear</i>		
HFMI S355, $L = 20$ m	<i>M-V-N interaction</i>		
HFMI S355, $L = 30$ m		<i>Deflection</i>	
HFMI S355, $L = 40$ m		<i>Deflection</i>	
HFMI S460, $L = 10$ m		<i>Deflection</i>	
HFMI S460, $L = 20$ m		<i>Deflection</i>	
HFMI S460, $L = 30$ m		<i>Deflection</i>	
HFMI S460, $L = 40$ m		<i>Deflection</i>	
HFMI S690, $L = 10$ m		<i>Deflection</i>	
HFMI S690, $L = 20$ m		<i>Deflection</i>	
HFMI S690, $L = 30$ m		<i>Deflection</i>	
HFMI S690, $L = 40$ m		<i>Deflection</i>	

A further observation from the results is that HFMI treatment essentially shifts the governing limit state. For the AW bridge, fatigue is the primary limiting factor for spans shorter than or equal to 30 m. When HFMI is applied, fatigue is eliminated as a governing factor. Instead, the design is limited by ULS for short spans with S355, and by SLS and deflection requirements for longer spans. Also worth mentioning is that all configurations using HSS (S460 and S690), as shown in Table 5.1 is governed by deflection. This shift confirms that HFMI successfully removes any limitations in design from fatigue, shifting the optimization challenge from service life of components towards the load-bearing capacity or flexural stiffness of the cross-section.

6

Conclusion

This study aimed to investigate whether High Frequency Mechanical Impact (HFMI) treatment can provide structural and economic benefits for simply supported, single track steel railway bridges across 10-40 m range of span lengths, and to evaluate whether the use of High Strength Steel (HSS) in combination with HFMI can further enhance these benefits. Limit state constraints included in the optimization were global bending, shear, interaction, flexural- and lateral torsional buckling, weld capacity, fatigue and deflection, where needed effective cross-section considering both plate buckling and shear lag was used. Apart from previously mentioned effects, eigenfrequency were also calculated and considered but not included as a constraint for the optimization. All considered limit state constraints was calculated and considered as stated in Eurocode. Parameters included in the Parametric Design Model (PDM) was cross-section parameters of main girders for each section, number of splices, placements of splices, profile sizes and throat thicknesses of longitudinal fillet weld. The PDM combined with a Genetic Algorithm (GA) in MATLAB was developed to optimize the geometry of the cross-section of bridges spanning 10-40 m, with the objective of minimizing total cost as the main goal. The cost based optimization considered expenses from investment, such as material, welding, HFMI, painting and manufacturing costs. Since the optimization was conducted using a GA, which is a stochastic optimization tool, each considered case was run at least three times to ensure that the global minimum of the design space was found. Based on the result and discussion presented in previous chapters, the following conclusions can be drawn:

- HFMI treatment shifts the governing limit state across all studied spans. In the As-Welded (AW) reference cases, fatigue governs all investigated spans. Once HFMI is applied, fatigue utilization drops by 40–80% and the governing limit state shifts to Ultimate Limit State (ULS) at the shorter span (10 m), to a combination of ULS and Serviceability Limit State (SLS) deflection at intermediate spans (20–30 m), and only to SLS deflection at 40 m.
- The shift in limit state towards SLS deflection implies limited benefits from the usage of HSS. This as a consequence of deflection only being affected by cross section and Young's modulus which not changes from improved steel grade. Even at shorter spans when there are some possibilities of improving ULS those are not great enough to counteract the higher unit prices of HSS.
- HFMI treatment delivers consistent reductions in material usage across the entire studied range. Steel consumption is reduced by 8–32% relative to the AW S355 baseline, for all combinations of span and steel grade. The largest mate-

rial savings are obtained at the shortest span (10 m, $\approx 31\%$), where fatigue has the greatest impact on the AW design. Material savings decreases gradually in parallel to increasing span as the AW baseline becomes increasingly stiffness dependent.

- Surface area is a significant but underexploited optimization target. Painting accounts for 20–36% of total build cost across all configurations, with HFMI treatment reducing painting cost by 2–20% for spans up to 30 m. This reduction comes from increased threshold level for stresses, allowing for lower or narrower cross sections and reduced painting cost. When evaluated over a full life cycle including periodic repainting, the realised benefit from HFMI is likely to be even greater than indicated by the investment cost presented in this thesis. This makes it an interesting factor to further investigate in future studies.
- The potential of HFMI treatment is independent of the fatigue assessment method employed. A direct comparison between optimized designs obtained under the Equivalent Stress Range (ESR) and the λ -method shows that HFMI delivers cost reductions in both cases, with the relative reductions being even larger under the λ -method. This is because the more conservative λ -method produces a heavier AW baseline against which the HFMI savings are measured. The absolute optimum geometry of the HFMI cases is essentially the same under both methods, since the deflection limit governs equally in both. The conclusion that HFMI treatment deliver benefits is therefore independent to the choice of fatigue assessment method.
- Geometric constraints have a critical impact on the profitability of both HFMI and HSS. The sensitivity study shows that limiting the beam height to the industry standard boundaries ($L/20$ – $L/30$) rather than the extended boundaries ($L/15$ – $L/40$) used in this study leads to a significant increase in total costs of 15–30%. This emphasises that maximizing the structural and economic benefits of HSS structures depends on allowing larger cross-sections to compensate for the critical bending limits.
- The governing limit states shift significantly depending on span length and HFMI application. It can be concluded from the discussion that the most decisive criterion for ULS is bending, shear and interaction for shorter bridges, while flange induced buckling becomes governing for longer spans. For SLS, deflection limits longer spans, while eigenfrequency is the critical factor for shorter AW bridges. FLS is governing for untreated bridges, but not for treated ones.

Suggestions for future studies:

- Investigate the impact of alternative traffic load compositions, such as a "Heavy Train Mix" with higher axle loads, to analyse how variations in used fatigue load model and magnitudes affect the structural efficiency of HFMI-treated designs.
- Expand the optimization framework to accommodate alternative cross-sectional topologies, such as single or double box girders or concrete-steel composite

structures, and evaluate their structural response against the pure steel configurations studied in this work.

- Integrate a full Life Cycle Cost (LCC) and Life Cycle Analysis (LCA) module directly into the objective function of the algorithm, shifting the optimization target from initial production costs to total sustainable and economic performance over the bridge's full lifespan.
- Conduct a more isolated and detailed investigation of how different geometric boundaries and girder height limitations affect the optimization efficiency, to further establish guidelines for when conventional design rules can be efficiently challenged.
- In this thesis ESR-method is used, which is a very calculation heavy method. It would be beneficial to evaluate the possibility of applying the conventional Palmgren-Miner (being less calculation heavy) damage accumulation method to HFMI-treated details. Since this thesis utilizes specific, verified fatigue assessment guidelines tailored for HFMI, investigating whether the more user-friendly and widely implemented Palmgren-Miner approach can yield comparable optimization results, would highly benefit practical industry applications.
- Investigate if a combination of HFMI and other Post Weld Treatment (PWT) methods (for example Bur Grinding (BG) or Tungsten Inert Gas (TIG) dressing) could further improve cost and material savings.
- Examine if there are cases where it is beneficial to HFMI treat details post bridge erection (on-site), to enable the possibility of ignoring self-weight effects in design.

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A

Appendix A

This appendix presents the manual calculations carried out at the start of the project in order to verify the PDM.

Design Bridge for Master Thesis

Dimensions

$$L := 10.6 \text{ m}$$

$$t_{fo} := 20 \text{ mm}$$

$$b_{fu} := 410 \text{ mm}$$

$$c := 1460 \text{ mm}$$

$$t_{fu} := 20 \text{ mm}$$

$$a_{bw} := 4 \text{ mm}$$

$$r := 1435 \text{ mm}$$

$$b_{fo} := 410 \text{ mm}$$

$$g := 9.82 \frac{\text{m}}{\text{s}^2}$$

$$h_w := 990 \text{ mm}$$

$$h_{rail} := 200 \text{ mm}$$

$$h_{sleeper} := 225 \text{ mm}$$

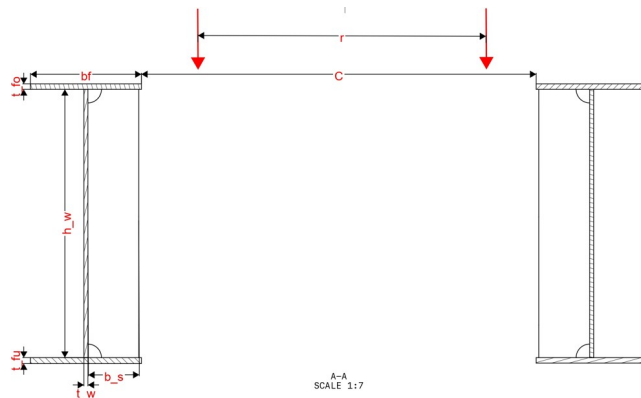
$$t_w := 15 \text{ mm}$$

$$h_{train} := 4000 \text{ mm}$$

$$c_{hb} := 1870 \cdot \text{mm}$$

$$c_1 := \frac{(b_{fo} + c - r)}{2} = 217.5 \text{ mm}$$

$$time_{in.use} := 120 \cdot \text{yr}$$



Material Parameters

S355

$$f_y := 355 \text{ MPa}$$

$$f_{y.345} := 345 \cdot \text{MPa}$$

EN 10025-2

$$\rho_{steel} := 7850 \frac{\text{kg}}{\text{m}^3}$$

$$\nu := 0.3$$

$$E := 210 \text{ GPa}$$

$$\varepsilon := \sqrt{\frac{235 \text{ MPa}}{f_y}} = 0.814$$

$$f_{u.S355} := 510 \text{ MPa}$$

From EN 1993
1-1 Table 3.1

Loads for LM71

$$q_{train.K} := 80 \frac{\text{kN}}{\text{m}}$$

$$q_{traction.K} := 33 \frac{\text{kN}}{\text{m}}$$

$$q_{w.K} := 0.64 \frac{\text{kN}}{\text{m}^2}$$

$$P_{train.K} := 250 \text{ kN}$$

$$P_{nosing.K} := 100 \text{ kN}$$

Safety Factors

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

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

$$\gamma_{M2} := 1.25$$

ULS Load Factors

$$\gamma_{Q.trac.main} := 1.5$$

$$\gamma_{Q.nosing.main} := 1.5$$

$$\gamma_{Q.LM71} := 1.5$$

$$\gamma_{Q.trac.sec} := 0.75$$

$$\gamma_{Q.nosing.sec} := 0.75$$

$$\gamma_{Q.wind} := 0.45$$

$$\gamma_{G.self} := 1.2$$

SLS Load Factors

$$\gamma_{Q.trac.main.SLS} := 1$$

$$\gamma_{Q.nosing.main.SLS} := 1$$

$$\gamma_{Q.LM71.SLS} := 1$$

$$\gamma_{Q.trac.sec.SLS} := 0.5$$

$$\gamma_{Q.nosing.sec.SLS} := 0.5$$

$$\gamma_{Q.wind.SLS} := 0.3$$

$$\gamma_{G.self.SLS} := 1$$

Preliminary Center of Gravity

$$A_{fo} := b_{fo} \cdot t_{fo} = 0.008 \text{ m}^2 \quad A_w := h_w \cdot t_w = 0.015 \text{ m}^2$$

$$A_{fu} := b_{fu} \cdot t_{fu} = 0.008 \text{ m}^2 \quad A_{tot} := 2 \cdot A_w + 2 \cdot A_{fu} + 2 \cdot A_{fo} = (6.25 \cdot 10^4) \text{ mm}^2$$

$$H_{tot.z} := t_{fo} + h_w + t_{fu} = 1.03 \text{ m} \quad H_{tot.y} := 2 \cdot b_{fo} + c = 2.28 \text{ m}$$

$$Y_{tp} := \frac{H_{tot.y}}{2} = 1.14 \text{ m}$$

$$Z_{tp} := \frac{2 \cdot A_{fo} \cdot \frac{t_{fo}}{2} + 2 \cdot A_w \cdot \left(t_{fo} + \frac{h_w}{2} \right) + 2 \cdot A_{fu} \cdot \left(\frac{t_{fu}}{2} + h_w + t_{fo} \right)}{A_{tot}} = 0.515 \text{ m}$$

Self Weights

Prescribed Weights

$$q_{railing} := 1 \frac{\text{kN}}{\text{m}} \quad q_{rails} := 1.7 \frac{\text{kN}}{\text{m}} \quad q_{grating} := 0.2 \frac{\text{kN}}{\text{m}^2} \cdot 1300 \text{ mm} = 0.26 \frac{\text{kN}}{\text{m}}$$

Material Data:

$$\text{IPE:} \quad \gamma_{IPE} := 0.22 \frac{\text{kN}}{\text{m}} \quad L_{IPE} := 5.5 \text{ m} \quad n_{IPE} := 7$$

$$\text{UPE:} \quad \gamma_{UPE} := 72.2 \frac{\text{kg}}{\text{m}} \cdot g = 0.709 \frac{\text{kN}}{\text{m}} \quad L_{UPE} := c = 1.46 \text{ m} \quad n_{UPE} := 4$$

$$\text{Sleepers:} \quad \gamma_{sleep} := 5.2 \frac{\text{kN}}{\text{m}^3} \quad L_{sleep} := 2.7 \text{ m} \quad n_{sleep} := 28$$

$$\text{L Profiles:} \quad \gamma_l := 15.5 \frac{\text{kg}}{\text{m}} \cdot g = 0.152 \frac{\text{kN}}{\text{m}} \quad L_l := 2070 \text{ mm} \quad n_l := 5$$

$$\text{Stiffening Plates:} \quad \gamma_{stiff.plate} := h_w \cdot 15 \text{ mm} \cdot 188 \text{ mm} \cdot \rho_{steel} \cdot g = 0.215 \frac{\text{kN}}{\text{m}} \quad n_{stiff} := 12$$

Total Self Weight

$$g_{I.girder} := A_{tot} \cdot \rho_{steel} \cdot g = 4.818 \frac{\text{kN}}{\text{m}}$$

$$q_{tot} := q_{railing} + q_{rails} + q_{grating} + \frac{\left(\gamma_{IPE} \cdot L_{IPE} \cdot n_{IPE} + \gamma_{UPE} \cdot L_{UPE} \cdot n_{UPE} + \gamma_{sleep} \cdot L_{sleep} \cdot n_{sleep} \cdot h_{sleep}^2 + \gamma_l \cdot L_l \cdot n_l + \gamma_{stiff.plate} \cdot n_{stiff} \right)}{L} + g_{I.girder} = 11.237 \frac{\text{kN}}{\text{m}}$$

Loads

$$\alpha_{traffic} := 1.33$$

$$L_{\phi} := L = 10.6 \text{ m}$$

$$e_1 := 0.05 \text{ m}$$

Load factor *TSFS 11, ch. 11§*
Determinant length

Eccentricity for vertical traffic loads (LM71 and SW/0)
is taken into account according to *TRVINFRA-00227*

Rail Shift Factor

$$\beta_{rs} := \frac{\frac{r}{2} + e_1}{r} = 0.535$$

$$\phi_{2.MG} := \frac{1.44}{\sqrt{\frac{L_{\phi}}{m} - 0.2}} + 0.82 = 1.291$$

Dynamic amplification factor.
SS-EN 1991-2 (based on $L_{\phi} = 10.6 \text{ m}$)

if $1 \leq \phi_{2.MG} \leq 1.67$ = "OK!"
|| "OK!"
else
|| "NOT OK!"

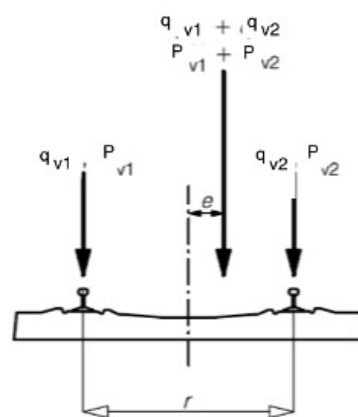
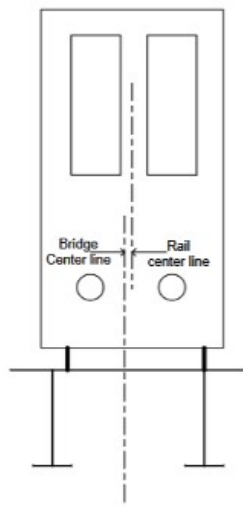


Figure 7

$$q_{v1}, q_{v2}, P_{v1}, P_{v2} = (1)$$

$$q_{v1} + q_{v2}, P_{v1} + P_{v2} = (2)$$

$$\frac{q_{v2}}{q_{v1}}, \frac{P_{v2}}{P_{v1}} \leq 1.25$$

$$e \leq \frac{r}{18}$$

$$r = (3)$$

Characteristic Static Load Effects

$$M_{max.train.K} := 2026.8 \text{ kN} \cdot \text{m}$$

For the whole section

$$V_{max.train.K} := 854.75 \text{ kN}$$

Total shear without eccentricity

$$M_{max.train.dyna.K} := M_{max.train.K} \cdot \alpha_{traffic} \cdot \phi_{2.MG} = 3.481 \text{ MN} \cdot \text{m}$$

With dynamic effects included

$$M_{max.train.e.K} := M_{max.train.dyna.K} \cdot (\beta_{rs}) = 1.862 \text{ MN} \cdot \text{m}$$

Critical case, including Rail Shift Factor

$$V_{max.train.dyna.K} := V_{max.train.K} \cdot \alpha_{traffic} \cdot \phi_{2.MG} = (1.468 \cdot 10^3) \text{ kN}$$

With dynamic effects included

$$V_{max.train.e.K} := V_{max.train.dyna.K} \cdot (\beta_{rs}) = 785.099 \text{ kN}$$

Max shear for worst side with eccentricity

Traction Forces

$$L_{traction} := L = 10.6 \text{ m}$$

$$q_{traction.K} := \begin{cases} L_{traction} \cdot q_{traction.K} < 1000 \text{ kN} \\ \frac{q_{traction.K}}{L_{traction}} \end{cases} = 33 \frac{\text{kN}}{\text{m}}$$

Traction load
Only included together with vertical traffic load, only multiplied by $\alpha_{traffic}$ and not $\phi_{2.MG}$!

$$N_{traction.K} := \frac{q_{traction.K} \cdot \alpha_{traffic} \cdot \frac{L}{2}}{2} = 116.309 \text{ kN}$$

Traction load in middle section

$$M_{traction} := N_{traction.K} \cdot (Z_{tp} + h_{sleeper}) = 86.068 \text{ kN} \cdot \text{m}$$

$$V_{traction} := \frac{q_{traction.K} \cdot \alpha_{traffic} \cdot (Z_{tp} + h_{sleeper})}{2} = 16.239 \text{ kN}$$

Wind Load

$$L_{train} := L = 10.6 \text{ m}$$

$$L_1 := 2120 \text{ mm}$$

Assumes train length is same as bridge
Length in one part between stiffener plates

$$b_{bridge} := L_{IPE} = 5.5 \text{ m} \quad d_{tot} := h_{train} + h_{rail} + h_{sleeper} + H_{tot.z} = 5.455 \text{ m}$$

$$\frac{b_{bridge}}{d_{tot}} = 1.008$$

This leads to

$$C_{fx} := 2.21$$

Figure 8.3 EN 1991-1-4

$$M_{W.train} := q_{w.K} \cdot h_{train} \cdot \left(\frac{\langle h_{train} \rangle}{2} + h_{rail} + h_{sleeper} + \frac{H_{tot.z}}{2} \right) \cdot C_{fx} = 16.633 \text{ kN} \cdot \text{m} \cdot \frac{1}{\text{m}}$$

Bending around
centre of
gravity of
main girders

$$M_{W.bridge} := q_{w.K} \cdot (H_{tot.z} + h_{rail} + h_{sleeper}) \cdot \left(\frac{(H_{tot.z} + h_{rail} + h_{sleeper})}{2} - (H_{tot.z} - Z_{tp}) \right) \cdot C_{fx} = 0.437 \text{ kN} \cdot \text{m} \cdot \frac{1}{\text{m}}$$

$$\Delta M_W := M_{W.train} + M_{W.bridge} = 17.071 \text{ kN} \cdot \text{m} \cdot \frac{1}{\text{m}} \quad Q_{long.W} := \frac{\Delta M_W}{(c + b_{fo})} = 9.129 \text{ kN} \cdot \frac{1}{\text{m}}$$

$$M_{long.W} := \frac{Q_{long.W} \cdot L^2}{8} = 128.213 \text{ kN} \cdot \text{m}$$

$$V_W := \frac{Q_{long.W} \cdot L}{2} = 48.382 \text{ kN}$$

$$N_{wind} := 47.87 \text{ kN}$$

Normal force from horizontal wind load transferred to trussers in
to the main girders

Wind Inputs from PointSketch

$$p_1 := C_{fx} \cdot q_{w.K} \cdot (H_{tot.z} + h_{rail} + h_{sleeper} + h_{train}) \cdot \frac{L_1}{2} = 8.178 \text{ kN}$$

Pointload at end

$$p_2 := C_{fx} \cdot q_{w.K} \cdot (H_{tot.z} + h_{rail} + h_{sleeper} + h_{train}) \cdot L_1 = 16.357 \text{ kN}$$

Pointload at middle

Loads from Self Weight

$$M_{max.self.K} := \frac{q_{tot} \cdot L^2}{2 \cdot 8} = 78.915 \text{ kN} \cdot \text{m}$$

$$V_{max.self.K} := \frac{q_{tot} \cdot L}{2 \cdot 2} = 29.779 \text{ kN}$$

Characteristic
maximum moment
and shear from
self weight

Loads from Nosing Force

$$P_{nosing} := 100 \text{ kN}$$

$$I_z := \frac{(t_{fo} + t_{fu}) \cdot b_{fo}^3}{12} + \frac{h_w \cdot t_w^3}{12} = (2.3 \cdot 10^8) \text{ mm}^4$$

$$W_{lat} := \frac{I_z}{\frac{b_{fo}}{2}} = (1.122 \cdot 10^6) \text{ mm}^3$$

$$M_{nosing} := \frac{P_{nosing} \cdot \alpha_{traffic} \cdot L_1}{2 \cdot 4} = 35.245 \text{ kN} \cdot \text{m}$$

Fixed between I-girders

$$N_{nosing} := 103.27 \cdot \alpha_{traffic} \cdot \text{kN} = 137.349 \text{ kN}$$

Normal force from horizontal nosing force transfered to trussers in to the main girders

Moment of Inertia (For One Beam) and Stresses

$$I_{prel} := \frac{b_{fo} \cdot t_{fo}^3}{12} + A_{fo} \cdot \left(Z_{tp} - \frac{t_{fo}}{2} \right)^2 + \frac{b_{fu} \cdot t_{fu}^3}{12} + A_{fu} \cdot \left(Z_{tp} - \left(H_{tot.z} - \frac{t_{fu}}{2} \right) \right)^2 + \frac{t_w \cdot h_w^3}{12} \downarrow = (5.396 \cdot 10^9) \text{ mm}^4$$

$$+ A_w \cdot \left(Z_{tp} - \frac{H_{tot.z}}{2} \right)^2$$

$$W_{top} := \frac{I_{prel}}{-Z_{tp}} = -1.048 \cdot 10^7 \text{ mm}^3 \quad W_{bot} := \frac{I_{prel}}{Z_{tp}} = (1.048 \cdot 10^7) \text{ mm}^3$$

$$\sigma_{top} := \frac{M_{max.train.e.K} \cdot \gamma_{Q.LM71} + M_{traction} \cdot \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} + M_{long.W} \cdot \gamma_{Q.wind} + M_{max.self.K} \cdot \gamma_{G.self}}{W_{top}} \downarrow = \begin{bmatrix} -326.519 \\ -344.422 \end{bmatrix} \text{ MPa}$$

$$- \frac{M_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix}}{W_{lat}} - \left(\frac{N_{traction.K} \cdot \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} \downarrow + N_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix} + N_{wind} \cdot \gamma_{Q.wind}}{\frac{A_{tot}}{2}} \right)$$

$$\sigma_{bot} := \frac{M_{max.train.e.K} \cdot \gamma_{Q.LM71} + M_{traction} \cdot \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} + M_{long.W} \cdot \gamma_{Q.wind} + M_{max.self.K} \cdot \gamma_{G.self}}{W_{bot}} \downarrow = \begin{bmatrix} 307.382 \\ 324.275 \end{bmatrix} \text{ MPa}$$

$$+ \frac{M_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix}}{W_{lat}} - \left(\frac{N_{traction.K} \cdot \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} \downarrow + N_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix} + N_{wind} \cdot \gamma_{Q.wind}}{\frac{A_{tot}}{2}} \right)$$

$$\psi_{prel} := \min \left(\frac{\sigma_{bot}}{\sigma_{top}} \right) = -0.942$$

Cross Sectional Class

Equations in this part are from EN 1993 1-5: Plated structural elements and EN 1993 1-1: General rules

Part 1: Outer Part of Top Flange

$$c_{p,1} := \frac{(b_{fo} - t_w)}{2} = 197.5 \text{ mm}$$

$$t_{p,1} := t_{fo} = 20 \text{ mm}$$

$\begin{aligned} &\text{if } 0 \leq \frac{c_{p,1}}{t_{p,1}} \leq 9 \cdot \varepsilon \\ &\quad \parallel \text{ "Class 1" } \\ &\text{else if } 9 \cdot \varepsilon < \frac{c_{p,1}}{t_{p,1}} \leq 10 \cdot \varepsilon \\ &\quad \parallel \text{ "Class 2" } \\ &\text{else if } 10 \cdot \varepsilon < \frac{c_{p,1}}{t_{p,1}} \leq 14 \cdot \varepsilon \\ &\quad \parallel \text{ "Class 3" } \\ &\text{else} \\ &\quad \parallel \text{ "Class 4" } \end{aligned}$	$= \text{"Class 3"}$	Since in class 3, it's OK
--	----------------------	---------------------------

Table 5.2 (sheet 2 of 3) -
EN 1993 1-1

Part 2: Web in I-Girder

Assuming Web to be in Class 4, Calculateing b_{eff} for Web

$$b_{slend} := h_w - a_{bw} \cdot \sqrt{2} = 984.343 \text{ mm}$$

Table 4.1 - EN 1993 1-5

$$k_\sigma := 7.81 - 6.29 \cdot \psi_{prel} + 9.78 \cdot \psi_{prel}^2 = 22.401$$

Table 4.1 - EN 1993 1-5

$$\lambda_p := \frac{\frac{b_{slend}}{t_w}}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = 0.6$$

$\begin{aligned} \rho_{eff} := &\text{if } (\lambda_p \leq 0.673) \\ &\parallel 1 \\ &\text{else if } (\lambda_p > 0.673) \wedge \left(\frac{\lambda_p - 0.055 \cdot (3 + \psi_{prel})}{\lambda_p^2} \leq 1 \right) \\ &\parallel \frac{\lambda_p - 0.055 \cdot (3 + \psi_{prel})}{\lambda_p^2} \\ &\text{else} \\ &\parallel 1 \end{aligned}$	$= 1$
---	-------

In class 3 or lower because
 λ_p is less than 0.673

eq 4.2

EN 1993 1-5

Check so the Assumption is Correct

$$c_{p,2} := h_w - \sqrt{2} \cdot a_{bw} = 984.343 \text{ mm} \quad t_{p,2} := t_w = 15 \text{ mm}$$

$$\alpha_1 := \max\left(\frac{\sigma_{top}}{(|\sigma_{bot}|)}\right) = -0.731$$

$$\text{if } 0 \leq \frac{c_{p,2}}{t_{p,2}} \leq \frac{396 \cdot \varepsilon}{13 \cdot \alpha_1 - 1}$$

|| "Class 1"

$$\text{else if } \frac{396 \cdot \varepsilon}{13 \cdot \alpha_1 - 1} < \frac{c_{p,2}}{t_{p,2}} \leq \frac{456 \cdot \varepsilon}{13 \cdot \alpha_1 - 1}$$

|| "Class 2"

$$\text{else if } \frac{456 \cdot \varepsilon}{13 \cdot \alpha_1 - 1} < \frac{c_{p,2}}{t_{p,2}} \leq 62 \cdot \varepsilon \cdot (1 - \psi_{prel}) \cdot \sqrt{-\psi_{prel}}$$

|| "Class 3"

else

|| "Class 4"

= "Class 3"

Ok assumption!

Table 5.2 (sheet 1 of 3) - *EN 1993 1-1*

Load Combination with ULS

$$Q_{d,ULS} = M_{max.train.e.K} \cdot \gamma_{train} + M_{traction} \begin{bmatrix} \gamma_{trac.main} \\ \gamma_{trac.sec} \end{bmatrix} + M_{long.W} \cdot \gamma_{wind} + M_{max.self.K} \cdot \gamma_{self} \downarrow \\ + M_{nosing} \cdot \begin{bmatrix} \gamma_{lat.sec} \\ \gamma_{lat.main} \end{bmatrix} + N_{traction.K} \cdot \begin{bmatrix} \gamma_{trac.main} \\ \gamma_{trac.sec} \end{bmatrix} + N_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix} + N_{wind} \cdot \gamma_{Q.wind}$$

$$\sigma_{top} := \frac{M_{max.train.e.K} \cdot \gamma_{Q.LM71} + M_{traction} \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} + M_{long.W} \cdot \gamma_{Q.wind} + M_{max.self.K} \cdot \gamma_{G.self}}{W_{top}} \downarrow = \begin{bmatrix} -326.519 \\ -344.422 \end{bmatrix} MPa$$

$$- \frac{M_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix}}{W_{lat}} \downarrow \\ - \left(\frac{N_{traction.K} \cdot \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} + N_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix} + N_{wind} \cdot \gamma_{Q.wind}}{\frac{A_{tot}}{2}} \right)$$

$$\sigma_{bot} := \frac{M_{max.train.e.K} \cdot \gamma_{Q.LM71} + M_{traction} \cdot \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} + M_{long.W} \cdot \gamma_{Q.wind} + M_{max.self.K} \cdot \gamma_{G.self}}{W_{bot}} \downarrow = \begin{bmatrix} 307.382 \\ 324.275 \end{bmatrix} MPa$$

$$+ \frac{M_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix}}{W_{lat}} \downarrow \\ - \left(\frac{N_{traction.K} \cdot \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} + N_{nosing} \cdot \begin{bmatrix} \gamma_{Q.nosing.sec} \\ \gamma_{Q.nosing.main} \end{bmatrix} + N_{wind} \cdot \gamma_{Q.wind}}{\frac{A_{tot}}{2}} \right)$$

$i := 0 \dots 1$

$$\begin{array}{|l} \text{if } \sigma_{top_i} \leq \frac{f_y}{\gamma_{M1}} \\ \quad \parallel \text{"OK!"} \\ \text{else if } \sigma_{top_i} > \frac{f_y}{\gamma_{M1}} \\ \quad \parallel \text{"NOT OK!"} \end{array} = \begin{bmatrix} \text{"OK!"} \\ \text{"OK!"} \end{bmatrix}$$

$$\begin{array}{|l} \text{if } \sigma_{bot_i} \leq \frac{f_y}{\gamma_{M1}} \\ \quad \parallel \text{"OK!"} \\ \text{else if } \sigma_{bot_i} > \frac{f_y}{\gamma_{M1}} \\ \quad \parallel \text{"NOT OK!"} \end{array} = \begin{bmatrix} \text{"OK!"} \\ \text{"OK!"} \end{bmatrix}$$

Compression Utilization Ratio Bending

$$\frac{\sigma_{top}}{f_{y.345}} = \begin{bmatrix} -94.64\% \\ -99.83\% \end{bmatrix}$$

$$\gamma_{M1}$$

Tension Utilization Ratio Bending

$$\frac{\sigma_{bot}}{f_{y.345}} = \begin{bmatrix} 89.1\% \\ 93.99\% \end{bmatrix}$$

$$\gamma_{M1}$$

Shear and Shear Buckling

Equations in this part are from EN 1993 1-5: Plated structural elements, otherwise it is specified

$$a_1 := 2120 \cdot \text{mm}$$

From initial design

$$\sigma_E := \frac{\pi^2 E \cdot t_w^2}{12 \cdot (1 - \nu^2) \cdot h_w^2} = 43.572 \text{ MPa}$$

eq. 4.8

$$k_{\tau sl} := 0$$

Due to no longitudinal stiffener on web plate

$$\eta := 1.2$$

For steel-classes up to and including S460

$$k_\tau := \begin{cases} \frac{a_1}{h_w} \geq 1 & \\ \left\| 5.34 + 4 \left(\frac{h_w}{a_1} \right)^2 + k_{\tau sl} \right\| & \\ \text{else if } \frac{a_1}{h_w} < 1 & \\ \left\| 4 + 5.34 \left(\frac{h_w}{a_1} \right)^2 + k_{\tau sl} \right\| & \end{cases} = 6.212$$

A.5

$$\tau_{cr} := k_\tau \cdot \sigma_E = 270.682 \text{ MPa}$$

eq. 5.4

$$\lambda_{w.1} := \frac{h_w}{37.4 \cdot t_w \cdot \varepsilon \cdot \sqrt{k_\tau}} = 0.87$$

For intermediate transverse stiffeners. eq. 5.6

Rigid End-Post

$$\chi_{w.main} := \begin{cases} \lambda_{w.1} < \frac{0.83}{\eta} & \\ \eta & \\ \text{else if } \frac{0.83}{\eta} \leq \lambda_{w.1} < 1.08 & \\ \frac{0.83}{\lambda_{w.1}} & \\ \text{else if } \lambda_{w.1} \geq 1.08 & \\ \frac{1.37}{0.7 + \lambda_{w.1}} & \end{cases} = 0.954$$

Table 5.1. Assumes transverse bulkhead provides rigid end-post

$$V_{bw.Rd.main} := \frac{\chi_{w.main} \cdot f_y \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 2.903 \text{ MN}$$

eq. 5.2

Shear Force Check

$$V_{ed.support} := \max \left(V_{max.train.e.K} \cdot \gamma_{Q.LM71} + V_{traction} \cdot \begin{bmatrix} \gamma_{Q.trac.main} \\ \gamma_{Q.trac.sec} \end{bmatrix} + V_{max.self.K} \cdot \gamma_{G.self} + V_W \cdot \gamma_{Q.wind} \right) = 1.26 \text{ MN}$$

if $V_{bw.Rd.main} \geq V_{ed.support}$ "OK!" else if $V_{bw.Rd.main} < V_{ed.support}$ "NOT OK!"	= "OK!"
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Utilization Ratio Shear, Only Considering Web Capacity

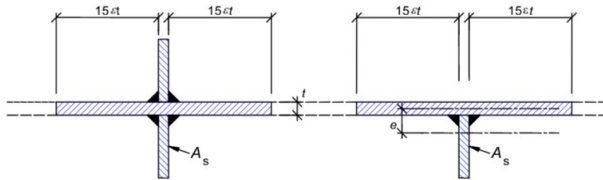
$$\frac{V_{ed.support}}{V_{bw.Rd.main}} = 43.39\%$$

if $\frac{V_{ed.support}}{V_{bw.Rd.main}} \leq 0.5$ "We don't need to check shear–bending interaction" else "We need to check shear–bending interaction"	= "We don't need to check shear–bending interaction"
---	--

From Chapter 7.1

Checking Minimum Requirements for Cross-Bracing

Equations in this part are from EN 1993 1-5: Plated structural elements



Figur 9.1 – Effektivt tvärsnitt för avstyvning

$$r_{notch} := 50 \cdot \text{mm} \quad \text{In long.weld}$$

$$t_s := 15 \cdot \text{mm}$$

$$L_{eff.w} := 2 \cdot 15 \cdot \varepsilon \cdot t_w + t_w = 381.127 \text{ mm}$$

$$h_{eff.s} := 188 \cdot \text{mm} - r_{notch} = 138 \text{ mm}$$

$$z_{tp.s} := \frac{L_{eff.w} \cdot t_w \cdot \frac{t_w}{2} + t_s \cdot h_{eff.s} \cdot \left(t_w + \frac{h_{eff.s}}{2} \right)}{L_{eff.w} \cdot t_w + t_s \cdot h_{eff.s}} = 27.836 \text{ mm}$$

$$I_{st} := \frac{t_s \cdot h_{eff.s}^3}{12} + \left(t_s \cdot h_{eff.s} \right) \cdot \left(z_{tp.s} - \left(t_w + \frac{h_{eff.s}}{2} \right) \right)^2 + \frac{L_{eff.w} \cdot t_w^3}{12} + \left(L_{eff.w} \cdot t_w \right) \cdot \left(z_{tp.s} - \left(\frac{t_w}{2} \right) \right)^2$$

$$I_{st.min} := \begin{cases} \text{if } \frac{a_1}{h_w} < \sqrt{2} \\ \left\| 1.5 \cdot h_w^3 \cdot t_s^3 \right\| \\ \text{else if } \frac{a_1}{h_w} \geq \sqrt{2} \\ \left\| 0.75 \cdot h_w \cdot t_s^3 \right\| \end{cases} = (2.506 \cdot 10^{-6}) \text{ m}^4 \quad \text{eq. 9.6}$$

Check so the Capacity for the Vertical Stiffeners is Enough

$$\begin{cases} \text{if } \frac{I_{st.min}}{I_{st}} \leq 1 \\ \left\| \text{"OK!"} \right\| \\ \text{else if } \frac{I_{st.min}}{I_{st}} > 1 \\ \left\| \text{"NOT OK!"} \right\| \end{cases} = \text{"OK!"}$$

Instability

Checking if girder is susceptible to LT-buckling, with general approach with the whole bridge length

Equations in this part are from EN 1993 1-1: General rules

$$L_{cr} := L = 10.6 \text{ m} \quad I_{f,z} := \frac{t_{fo} \cdot b_{fo}^3}{12} + \frac{(Z_{tp} - t_{fo}) \cdot \frac{1}{3} \cdot t_w^3}{12} = (1.149 \cdot 10^8) \text{ mm}^4$$

$$A_{f,eff} := t_{fo} \cdot b_{fo} + (Z_{tp} - t_{fo}) \cdot \frac{1}{3} \cdot t_w = (1.068 \cdot 10^4) \text{ mm}^2$$

$$I_{f,z,OB} := I_{f,z} + A_{f,eff} \cdot \left(\frac{c_{hb}}{2} \right)^2 = (9.447 \cdot 10^9) \text{ mm}^4$$

OB = One Beam

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_{f,z,OB}}{L_{cr}^2} = 174.266 \text{ MN}$$

$$N_{Rk} := \frac{A_{tot}}{2} \cdot f_y = 11.094 \text{ MN}$$

$$N_{Ed} := \max \left(N_{traction,K} \cdot \left[\frac{\gamma_{Q,traction,main}}{\gamma_{Q,traction,sec}} \right] + N_{nosing} \cdot \left[\frac{\gamma_{Q,nosing,sec}}{\gamma_{Q,nosing,main}} \right] + N_{wind} \cdot \gamma_{Q,wind} \right) = 314.797 \text{ kN}$$

$$M_{y,Rk} := -W_{top} \cdot f_y = 3.719 \text{ MN} \cdot \text{m}$$

$$M_{y,Ed} := \max \left(M_{max,train,e,K} \cdot \gamma_{Q,LM71} + M_{traction} \left[\frac{\gamma_{Q,traction,main}}{\gamma_{Q,traction,sec}} \right] + M_{long,W} \cdot \gamma_{Q,wind} + M_{max,self,K} \cdot \gamma_{G,self} \right) = 3.074 \text{ MN} \cdot \text{m}$$

$$n_{komb} := \frac{N_{Ed}}{N_{Rk}} = 0.028 \quad m_{komb} := \frac{M_{y,Ed}}{M_{y,Rk}} = 0.826$$

$$\alpha_{ult,k} := \frac{1}{n_{komb} + m_{komb}} = 1.17$$

Chapter 6.3.4 (4)

$$\alpha_{cr,op} := \frac{N_{cr}}{N_{Ed}} = 553.584$$

$$\lambda_{op} := \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} = 0.046$$

eq. 6.64

$$\alpha_{ib} := 0.49$$

Table 6.1 curve c

$$\Phi_{HB} := 0.5 \cdot (1 + \alpha_{ib} \cdot (\lambda_{op} - 0.2) + \lambda_{op}^2) = 0.463$$

eq. 6.56

$$\chi_{HB} := \frac{1}{\Phi_{HB} + \sqrt{\Phi_{HB}^2 - \lambda_{op}^2}} = 1.082$$

LT-Buckling

$$\alpha_{LT} := 0.76$$

$$\Phi_{LT} := 0.5 \cdot (1 + \alpha_{LT} \cdot (\lambda_{op} - 0.2) + \lambda_{op}^2) = 0.443$$

$$\chi_{LT} := \min \left(1, \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \lambda_{op}^2}} \right) = 1$$

$$k_c := 0.9$$

$$f := \begin{cases} 1 \geq 1 - 0.5 \cdot (1 - k_c) \cdot (1 - 2.0 \cdot (\lambda_{op} - 0.8)^2) \\ \quad \parallel 1 \geq 1 - 0.5 \cdot (1 - k_c) \cdot (1 - 2.0 \cdot (\lambda_{op} - 0.8)^2) \\ \text{else} \\ \quad \parallel 1 \end{cases} = 1$$

$$\chi_{LT.mod} := \min \left(1, \frac{\chi_{LT}}{f} \right) = 1$$

$$\chi_{res} := \min \left(1, \frac{n_{komb} \cdot \chi_{HB} + m_{komb} \cdot \chi_{LT.mod}}{n_{komb} + m_{komb}} \right) = 1$$

Check for Instability with LT-Buckling

$$\frac{\chi_{res} \cdot \alpha_{ult.k}}{\gamma_{M1}} = 1.17$$

$\begin{aligned} &\text{if } \frac{\chi_{res} \cdot \alpha_{ult.k}}{\gamma_{M1}} \geq 1 \\ &\quad \parallel \text{"OK for instability"} \\ &\text{else} \\ &\quad \parallel \text{"NOT OK for instability"} \end{aligned}$	= "OK for instability"
--	------------------------

Table 6.3 curve d

eq. 6.56

eq. 6.56

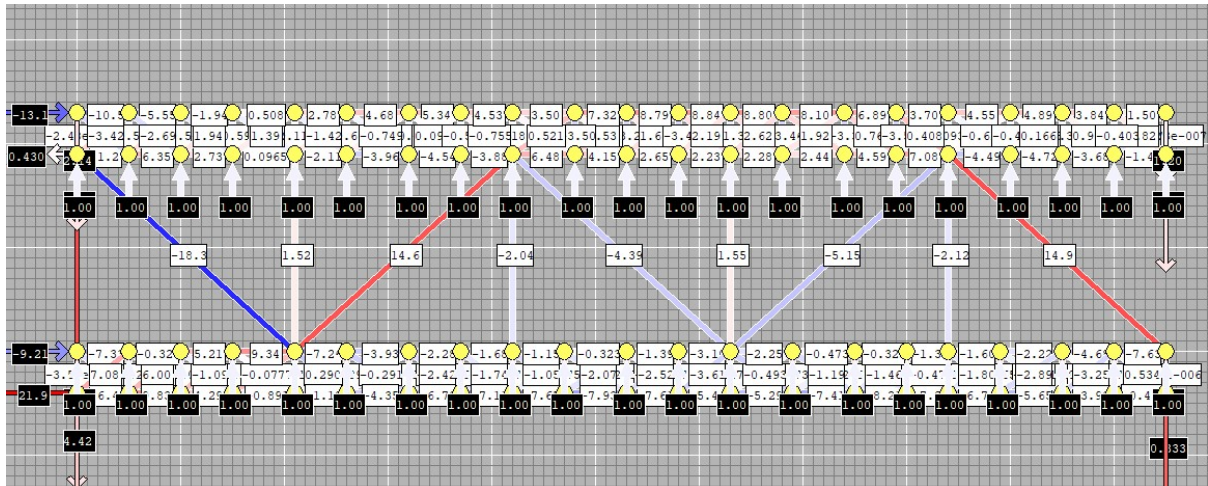
Table 6.6

eq. 6.58

eq. 6.63

Calculate Forces (From PointSketch)

Wind Force

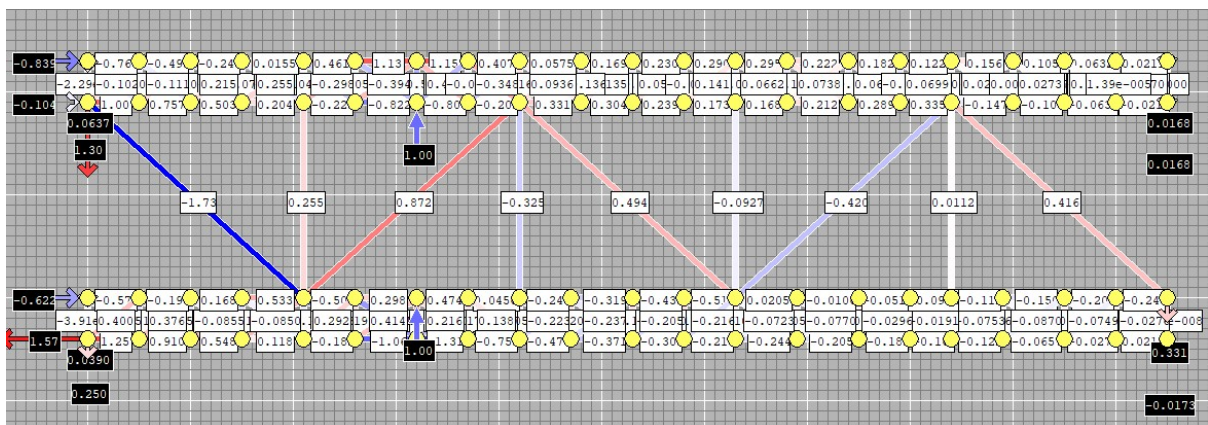


Unit loads of 1 has been appllied in PointSketch which gives unitforces of:

$$N_{unit.L.wind} := 18.3 \cdot m \quad N_{unit.UPE.wind} := 2.12 \cdot m$$

$$q_{wind.H} := \frac{q_{w.K} \cdot (h_{sleeper} + h_{train} + h_{rail} + H_{tot.z})}{2} = 1.746 \frac{kN}{m}$$

Nosing Force L-Profile (Truss-System)

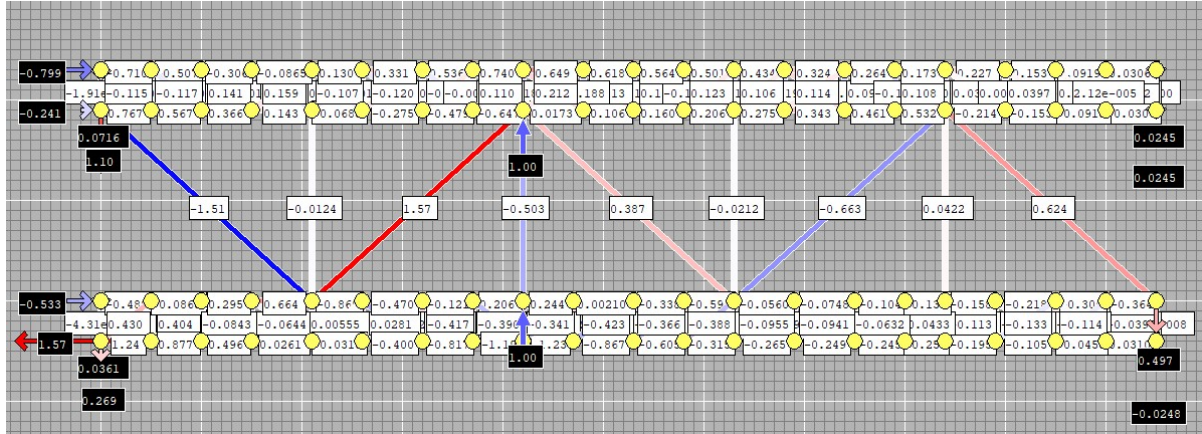


Unit loads of 1 has been appllied in PointSketch which gives unitforces of:

$$N_{unit.L.nosing} := 1.73$$

Each unit load corresponds to half of the nosing force (50kN)

Nosing Force UPE-Profile



Unit loads of 1 has been applied in PointSketch which gives unitforces of:

$$N_{unit.UPE.nosing} := 0.503$$

Each unit load corresponds to half of the nosing force (50kN)

L-Profile (Truss-System)

$$N_{fack.wind} := N_{unit.L.wind} \cdot q_{wind.H} \cdot \gamma_{Q.wind} = 14.375 \text{ kN}$$

$$N_{fack.nosing} := N_{unit.L.nosing} \cdot \frac{P_{nosing}}{2} \cdot \gamma_{Q.nosing.main} = 129.75 \text{ kN}$$

$$N_{Ed,fack} := N_{fack.nosing} \cdot \alpha_{traffic} + N_{fack.wind} = 186.943 \text{ kN}$$

UPE-Profile

$$N_{UPE.wind} := N_{unit.UPE.wind} \cdot q_{wind.H} \cdot \gamma_{Q.wind} = 1.665 \text{ kN}$$

$$N_{UPE.nosing} := N_{unit.UPE.nosing} \cdot \frac{P_{nosing}}{2} \cdot \gamma_{Q.nosing.main} = 37.725 \text{ kN}$$

$$N_{Ed.UPE} := (N_{UPE.nosing} \cdot \alpha_{traffic} + N_{UPE.wind}) = 51.84 \text{ kN}$$

L-Profile Truss-System

$$L_{cr.fack} := \frac{0.9 \cdot \sqrt{c_{hb}^2 + a_1^2}}{c_{hb}} \cdot c = 1.986 \text{ m}$$

$$I_{z.L} := 90.2 \cdot 10^4 \text{ mm}^4 \quad A_L := 1970 \cdot \text{mm}^2$$

$$\alpha_{fack} := 0.34$$

$$N_{cr.fack} := \frac{\pi^2 \cdot E \cdot I_{z.L}}{L_{cr.fack}^2} = 473.806 \text{ kN}$$

$$\lambda_{fack} := \sqrt{\frac{A_L \cdot f_y}{N_{cr.fack}}} = 1.215$$

From Tibnor's structure tables
Buckling curve B for all L-profiles

eq. 6.52

$$\Phi_{fack} := 0.5 \cdot (1 + \alpha_{fack} \cdot (\lambda_{fack} - 0.2) + \lambda_{fack}^2) = 1.411$$

$$\chi_{fack} := \min \left(1, \frac{1}{\Phi_{fack} + \sqrt{\Phi_{fack}^2 - \lambda_{fack}^2}} \right) = 0.47$$

eq. 6.56

$$N_{b.Rd.fack} := \frac{\chi_{fack} \cdot A_L \cdot f_y}{\gamma_{M1}} = 328.762 \text{ kN}$$

Check for Instability in Truss-System

if $\frac{N_{Ed.fack}}{N_{b.Rd.fack}} \leq 1$ "OK for instability" else "NOT OK for instability"	= "OK for instability"
---	------------------------

Utilization Ratio Truss-System

$$\frac{N_{Ed.fack}}{N_{b.Rd.fack}} = 56.86\%$$

UPE-Profile

$$L_{cr,UPE} := 1 \cdot c_{hb} = 1.87 \text{ m}$$

Assuming that cross beams can buckled with length between main girder webs

$$I_{z,UPE} := 1040 \cdot 10^4 \cdot \text{mm}^4 \quad A_{UPE} := 9190 \cdot \text{mm}^2$$

From Tibnor's structure tables

$$\alpha_{UPE} := 0.49$$

Buckling curve C for all UPE-profiles

$$N_{cr,UPE} := \frac{\pi^2 \cdot E \cdot I_{z,UPE}}{L_{cr,UPE}^2} = (6.164 \cdot 10^3) \text{ kN}$$

$$\lambda_{UPE} := \sqrt{\frac{A_{UPE} \cdot f_y}{N_{cr,UPE}}} = 0.728$$

eq. 6.52

$$\Phi_{UPE} := 0.5 \cdot (1 + \alpha_{UPE} \cdot (\lambda_{UPE} - 0.2) + \lambda_{UPE}^2) = 0.894$$

$$\chi_{UPE} := \min \left(1, \frac{1}{\Phi_{UPE} + \sqrt{\Phi_{UPE}^2 - \lambda_{UPE}^2}} \right) = 0.708$$

$$N_{b,Rd,UPE} := \frac{\chi_{UPE} \cdot A_{UPE} \cdot f_y}{\gamma_{M1}} = (2.308 \cdot 10^3) \text{ kN}$$

Check for Instability in UPE-Profile

if $\frac{N_{Ed,UPE}}{N_{b,Rd,UPE}} \leq 1$ "OK for instability" else "NOT OK for instability"	= "OK for instability"
---	------------------------

Utilization Ratio UPE-Profile

$$\frac{N_{Ed,UPE}}{N_{b,Rd,UPE}} = 2.25\%$$

Local Instability

$$k_{c1} := 0.9$$

Table 6.6

$$I_{f,z} := \frac{t_{fo} \cdot b_{fo}^3}{12} + \frac{(Z_{tp} - t_{fo}) \cdot \frac{1}{3} \cdot t_w^3}{12} = (1.149 \cdot 10^8) \text{ mm}^4$$

$$A_{f,eff} := t_{fo} \cdot b_{fo} + (Z_{tp} - t_{fo}) \cdot \frac{1}{3} \cdot t_w = (1.068 \cdot 10^4) \text{ mm}^2$$

$$L_c := a_1 = 2.12 \text{ m} \quad i_{f,z} := \sqrt{\frac{I_{f,z}}{A_{f,eff}}} = 103.754 \text{ mm}$$

Chapter 6.3.2.4

$$\lambda_{1,local} := \pi \cdot \sqrt{\frac{E}{f_y}} = 76.409$$

Chapter 6.3.2.4

$$\lambda_f := \frac{k_c \cdot L_c}{i_{f,z} \cdot \lambda_{1,local}} = 0.241$$

eq. 6.59

$$\lambda_{LT,0} := 0.4$$

Chapter 6.3.2.3

$$\lambda_{c,0} := \lambda_{LT,0} + 0.1 = 0.5$$

Chapter 6.3.2.4

$$\sigma_{c,Rd} := \frac{f_y}{\gamma_{M1}} = 355 \text{ MPa}$$

$$\sigma_{ed} := \max(-\sigma_{top}) = 344.422 \text{ MPa}$$

Only checking top flange, since it is the worst case and only top flange that can buckle since it's in compression.

$\text{if } \lambda_f \leq \lambda_{c,0} \cdot \frac{\sigma_{c,Rd}}{\sigma_{ed}}$	$= \text{"Local LT - buckling doesn't need to be checked!"}$
$\parallel \text{"Local LT - buckling doesn't need to be checked!"}$	
else	
$\parallel \text{"Need to check local LT - buckling!"}$	eq. 6.59

"Utilization" Ratio Local Instability

$$\frac{\lambda_f}{\lambda_{c,0} \cdot \frac{\sigma_{c,Rd}}{\sigma_{ed}}} = 46.7\%$$

Checking Shear in Longitudinal Welding

Static moment (through centre of gravity, main girders)

Equations in this part are from EN 1993 1-8: Design of joints

$$S_{ztp.top} := A_{fo} \cdot \left(Z_{tp} - \frac{t_{fo}}{2} \right) = -4.141 \cdot 10^6 \text{ mm}^3$$

$$S_{ztp.bot.sym} := A_{fu} \cdot \left(H_{tot.z} - Z_{tp} - \frac{t_{fu}}{2} \right) = (4.141 \cdot 10^6) \text{ mm}^3$$

$$I_y := I_{prel} = (5.396 \cdot 10^9) \text{ mm}^4$$

$$b_{bot} := 2 \cdot a_{bw} = 8 \text{ mm}$$

$$V_{ed.support} = (1.26 \cdot 10^3) \text{ kN}$$

$$\beta_w := 0.9$$

Table 4.1

In top we check web since the butt welds will have more capacity than the web.
In the bottom we check the welds since fillet welds will have less capacity (or equal capacity):

Fillet Weld Between Bottom Flange and Web

$$\tau_{11.bottom.weld} := \frac{V_{ed.support} \cdot S_{ztp.bot.sym}}{I_y \cdot b_{bot}} = 120.826 \text{ MPa}$$

$$\sigma_{T.fw} := 0 \cdot \text{MPa} \quad \tau_{T.fw} := \sigma_{T.fw} = 0 \text{ MPa}$$

$\text{if } \sqrt{\sigma_{T.fw}^2 + 3 \cdot (\tau_{T.fw}^2 + \tau_{11.bottom.weld}^2)} \leq \frac{f_{u.S355}}{(\beta_w \cdot \gamma_{M2})} \wedge \sigma_{T.fw} \leq \frac{0.9 \cdot f_{u.S355}}{\gamma_{M2}} = \text{"OK resistance of the fillet weld"}$	$= \text{"OK resistance of the fillet weld"}$
else	
$\text{"NOT OK resistance of the fillet weld"}$	

eq. 4.1

Utilization Ratio Bottom Weld

$$\frac{\sqrt{\sigma_{T.fw}^2 + 3 \cdot (\tau_{T.fw}^2 + \tau_{11.bottom.weld}^2)}}{\frac{f_{u.S355}}{(\beta_w \cdot \gamma_{M2})}} = 46.16\%$$

Butt Weld Between Top Flange and Web

$$b_{top} := t_w = 15 \text{ mm}$$

$$\tau_{11.top.weld} := \frac{V_{ed.support} \cdot S_{ztp.top}}{I_y \cdot b_{top}} = -64.44 \text{ MPa}$$

Since it is a butt weld the web is the weak part

$$\sigma_{T.bw} := 0 \text{ MPa}$$

$$\tau_{T.bw} := \sigma_{T.bw} = 0 \text{ MPa}$$

$\text{if } \sqrt{\sigma_{T.bw}^2 + 3 \cdot (\tau_{T.bw}^2 + \tau_{11.top.weld}^2)} \leq \frac{f_y}{\gamma_{M0}} \wedge \sigma_{T.bw} \leq \frac{0.9 \cdot f_y}{\gamma_{M0}} = \text{"OK resistance of the butt weld"}$	$eq. 4.1$
$\parallel \text{"OK resistance of the butt weld"}$	
else	
$\parallel \text{"NOT OK resistance of the butt weld"}$	

Utilization Ratio Top Weld

$$\frac{\sqrt{\sigma_{T.bw}^2 + 3 \cdot (\tau_{T.bw}^2 + \tau_{11.top.weld}^2)}}{\frac{f_y}{\gamma_{M0}}} = 31.4\%$$

Web Check From Local Patch Load

Equations in this part are from EN 1993 1-5: Plated structural elements

$$a_1 = 2.12 \text{ m} \quad f_{y,w} := f_{y,345} = 345 \text{ MPa}$$

$$h_w = 990 \text{ mm} \quad s_s := h_{\text{sleeper}} = 0.225 \text{ m}$$

$$k_F := 6 + 2 \cdot \left(\frac{h_w}{a_1} \right)^2 = 6.436$$

$$m_1 := \frac{f_{y,345} \cdot b_{fo}}{f_{y,345} \cdot t_w} = 27.333$$

$$F_{cr} := 0.9 \cdot k_F \cdot E \cdot \frac{t_w^3}{h_w} = (4.147 \cdot 10^3) \text{ kN}$$

$$l_y(m_{2,1}) := s_s + 2 \cdot t_{fo} \cdot \left(1 + \sqrt{m_1 + m_{2,1}} \right)$$

$$\lambda_F(m_{2,1}) := \sqrt{\frac{l_y(m_{2,1}) \cdot t_w \cdot f_{y,345}}{F_{cr}}}$$

Constraint Values	$m_{2,1} := 1$
	$m_{2,1} = \begin{cases} \lambda_F(m_{2,1}) > 0.5 \\ \left\ 0.02 \cdot \left(\frac{h_w}{t_{fo}} \right)^2 \right\ \\ \text{else} \\ 0 \end{cases}$
Solver	$m_2 := \text{find}(m_{2,1}) = 49.005$

$$\lambda_F(m_2) = 0.876 \quad l_y(m_2) = 614.487 \text{ mm}$$

$$\chi_F := \begin{cases} \frac{0.5}{\lambda_F(m_2)} < 1 \\ \frac{0.5}{\lambda_F(m_2)} \end{cases} = 0.571 \quad \text{eq. 6.3}$$

$$L_{eff} := l_y(m_2) \cdot \chi_F = 350.86 \text{ mm} \quad \text{eq. 6.2}$$

$$P_{patch} := P_{train,K} \cdot \phi_{2,MG} \cdot \alpha_{traffic} \cdot \beta_{rs} = 229.628 \text{ kN}$$

$$F_{Ed} := P_{patch} \quad F_{Rd} := \frac{f_{y,w} \cdot L_{eff} \cdot t_w}{\gamma_{M1}} = (1.816 \cdot 10^3) \text{ kN} \quad \text{eq. 6.1}$$

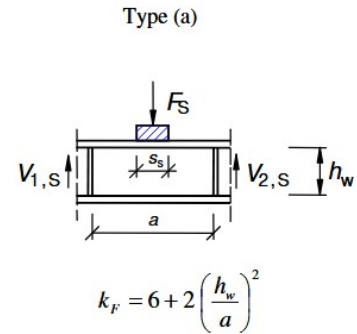
$$\eta_2 := \frac{F_{Ed}}{\frac{f_{y,w} \cdot L_{eff} \cdot t_w}{\gamma_{M1}}}$$

if $\eta_2 \leq 1$	= "OK resistance of web"
"OK resistance of web"	
else	
"NOT OK resistance of web"	eq. 6.14

Utilization of Local Web Capacity

$$\eta_2 = 12.6\%$$

Figure 6.1, Type(a)



Resistance of Cross-Sections; Yield Criterion

Equations in this part are from EN 1993 1-1: General rules

$$\sigma_{x.Ed} := \frac{M_{max.train.e.K} \cdot \gamma_{Q.LM71} + M_{traction} \left[\frac{\gamma_{Q.trac.main}}{\gamma_{Q.trac.sec}} \right] + M_{long.W} \cdot \gamma_{Q.wind} + M_{max.self.K} \cdot \gamma_{G.self}}{W_{top}} \downarrow = \begin{bmatrix} -302.96 \\ -297.304 \end{bmatrix} \text{ MPa}$$

$$- \left(\frac{N_{traction.K} \cdot \left[\frac{\gamma_{Q.trac.main}}{\gamma_{Q.trac.sec}} \right] + N_{nosing} \cdot \left[\frac{\gamma_{Q.nosing.sec}}{\gamma_{Q.nosing.main}} \right] + N_{wind} \cdot \gamma_{Q.wind}}{\frac{A_{tot}}{2}} \right)$$

$$\sigma_{x.Ed} := \max(\sigma_{x.Ed}) = -297.304 \text{ MPa}$$

$$\sigma_{z.Ed} := \frac{P_{patch}}{L_{eff} \cdot t_{fo}} = 32.724 \text{ MPa}$$

$$\tau_{Ed} := \tau_{11.top.weld} = -64.44 \text{ MPa}$$

$$\text{if } \left(\frac{\sigma_{x.Ed}}{\gamma_{M0} f_y} \right)^2 + \left(\frac{\sigma_{z.Ed}}{\gamma_{M0} f_y} \right)^2 - \left(\frac{\sigma_{x.Ed}}{\gamma_{M0} f_y} \right) \cdot \left(\frac{\sigma_{z.Ed}}{\gamma_{M0} f_y} \right) + 3 \cdot \left(\frac{\tau_{Ed}}{\gamma_{M0} f_y} \right)^2 \leq 1 \quad \text{“OK, meets the criterion for yield”} \quad \text{eq. 6.1}$$

|| “OK, meets the criterion for yield”

else

|| “NOT OK, doesn't meet the criterion for yield”

Utilization Ratio Von Misses

$$\left(\frac{\sigma_{x.Ed}}{\gamma_{M0} f_y} \right)^2 + \left(\frac{\sigma_{z.Ed}}{\gamma_{M0} f_y} \right)^2 - \left(\frac{\sigma_{x.Ed}}{\gamma_{M0} f_y} \right) \cdot \left(\frac{\sigma_{z.Ed}}{\gamma_{M0} f_y} \right) + 3 \cdot \left(\frac{\tau_{Ed}}{\gamma_{M0} f_y} \right)^2 = 88.59\%$$

Load Combination with SLS

$$Q_{d.SLS} = G + Q_{main} + \sum \psi_{other} \cdot Q_{other} \quad SLS$$

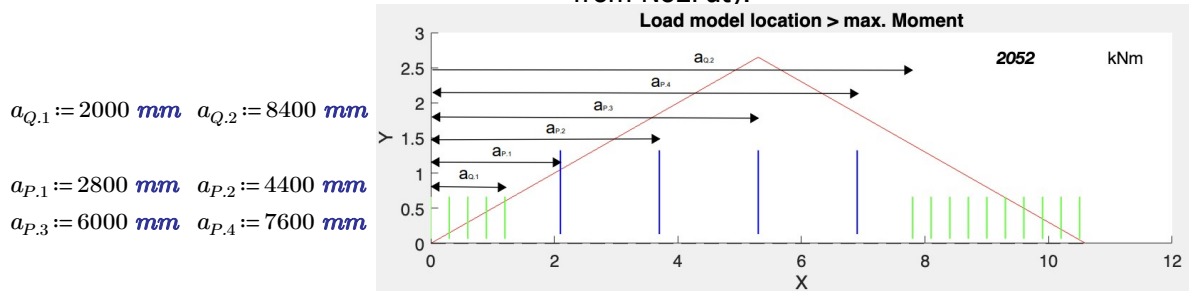
$$q_{train.beta.K} := 80 \cdot \beta_{rs} \cdot \frac{kN}{m} \quad \text{For one I-girder}$$

$$q_{tot} = 11.237 \frac{kN}{m}$$

$$Q_{long.W} = 9.129 \frac{kN}{m}$$

$$P_{train.beta.K} := 250 \cdot \beta_{rs} \cdot kN \quad \text{For one I-girder}$$

Train loads is main load (load placement from No2Fat):



$$p_{train.Q.1} := \alpha_{traffic} \cdot q_{train.beta.K} \cdot \frac{a_{Q.1}^2 \cdot L^2}{96 \cdot E \cdot I_y} \cdot \left(3 - \frac{2 \cdot a_{Q.1}^2}{L^2} \right) \cdot \gamma_{Q.LM71.SLS} = 0.689 \text{ mm}$$

$$p_{train.Q.2} := \alpha_{traffic} \cdot q_{train.beta.K} \cdot \frac{(L - a_{Q.2})^2 \cdot L^2}{96 \cdot E \cdot I_y} \cdot \left(3 - \frac{2 \cdot (L - a_{Q.2})^2}{L^2} \right) \cdot \gamma_{Q.LM71.SLS} = 0.829 \text{ mm}$$

$$p_{train.P.1} := \alpha_{traffic} \cdot P_{train.beta.K} \cdot \frac{a_{P.1} \cdot L^2}{48 \cdot E \cdot I_y} \cdot \left(3 - \frac{4 \cdot a_{P.1}^2}{L^2} \right) \cdot \gamma_{Q.LM71.SLS} = 2.799 \text{ mm}$$

$$p_{train.P.2} := \alpha_{traffic} \cdot P_{train.beta.K} \cdot \frac{a_{P.2} \cdot L^2}{48 \cdot E \cdot I_y} \cdot \left(3 - \frac{4 \cdot a_{P.2}^2}{L^2} \right) \cdot \gamma_{Q.LM71.SLS} = 3.735 \text{ mm}$$

$$p_{train.P.3} := \alpha_{traffic} \cdot P_{train.beta.K} \cdot \frac{a_{P.3} \cdot L^2}{48 \cdot E \cdot I_y} \cdot \left(3 - \frac{4 \cdot a_{P.3}^2}{L^2} \right) \cdot \gamma_{Q.LM71.SLS} = 3.788 \text{ mm}$$

$$p_{train.P.4} := \alpha_{traffic} \cdot P_{train.beta.K} \cdot \frac{(L - a_{P.4}) \cdot L^2}{48 \cdot E \cdot I_y} \cdot \left(3 - \frac{4 \cdot (L - a_{P.4})^2}{L^2} \right) \cdot \gamma_{Q.LM71.SLS} = 2.953 \text{ mm}$$

$$p_{self} := \frac{q_{tot}}{2} \cdot \frac{5 \cdot L^4}{384 \cdot E \cdot I_y} \cdot \gamma_{G.self.SLS} = 0.815 \text{ mm}$$

Half of the self-weight in p_{self}

$$p_{wind} := Q_{long.W} \cdot \frac{5 \cdot L^4}{384 \cdot E \cdot I_y} \cdot \gamma_{Q.wind.SLS} = 0.397 \text{ mm}$$

$$p_{tot} := p_{train.Q.1} + p_{train.Q.2} + p_{train.P.1} + p_{train.P.2} + p_{train.P.3} + p_{train.P.4} + p_{self} + p_{wind} = 16.005 \text{ mm}$$

$$p_{allowed} := \frac{L}{600} = 17.667 \text{ mm}$$

if $p_{tot} \leq p_{allowed}$	= "Deflection is OK!"
"Deflection is OK!"	
else	
"Deflection is NOT OK!"	

Eigenfrequency

Equations in this part are from EN 1991 1-2: Traffic loads on bridges

$$n_{0,over} := 94.76 \cdot \left(\frac{L}{m} \right)^{-0.748} = 16.207 \text{ [Hz]} \quad \text{Table 6.10}$$

$$n_{0,under} := \begin{cases} \frac{80}{L} \cdot m & \text{if } 4 \cdot m \leq L \leq 20 \cdot m \\ 23.58 \cdot \left(\frac{L}{m} \right)^{-0.592} & \text{else if } 20 \cdot m < L \leq 100 \cdot m \end{cases} = 7.547 \text{ [Hz]} \quad \text{Table 6.10}$$

$$\delta_0 := \frac{5 \cdot \frac{q_{tot}}{2} \cdot L^4}{384 \cdot E \cdot I_y} = 0.815 \text{ mm} \quad \text{Standard cases}$$

$$n_0 := \frac{17.75}{\sqrt{\frac{\delta_0}{mm}}} = 19.66 \text{ [Hz]} \quad \text{eq. 6.3}$$

$$\begin{cases} \text{if } n_{0,under} \leq n_0 \leq n_{0,over} & = \text{"NOT OK!"} \\ \text{||} & \\ \text{"OK!"} & \\ \text{else} & \\ \text{||} & \\ \text{"NOT OK!"} & \end{cases}$$

Utilization Rates

$$\frac{\sigma_{bot}}{\frac{f_{y.345}}{\gamma_{M1}}} = \begin{bmatrix} 89.1\% \\ 93.99\% \end{bmatrix}$$

Tension utilization ratio
bending

$$\frac{\sigma_{top}}{\frac{f_{y.345}}{\gamma_{M1}}} = \begin{bmatrix} -94.64\% \\ -99.83\% \end{bmatrix}$$

Compression utilization ratio
bending

$$\frac{V_{ed.support}}{V_{bw.Rd.main}} = 43.39\%$$

Utilization ratio shear only
considering web capacity

$$\frac{N_{Ed.fack}}{N_{b.Rd.fack}} = 56.86\%$$

Utilization ratio truss-system

$$\frac{N_{Ed.UPE}}{N_{b.Rd.UPE}} = 2.25\%$$

Utilization ratio UPE-profile

$$\frac{\lambda_f}{\lambda_{c.0} \cdot \frac{\sigma_{c.Rd}}{\sigma_{ed}}} = 46.7\%$$

"Utilization" ratio local instability

$$\frac{\sqrt{\sigma_{T.fw}^2 + 3 \cdot (\tau_{T.fw}^2 + \tau_{11.bottom.weld}^2)}}{\frac{f_{u.S355}}{(\beta_w \cdot \gamma_{M2})}} = 46.2\%$$

Utilization ratio bottom weld

$$\frac{\sqrt{\sigma_{T.bw}^2 + 3 \cdot (\tau_{T.bw}^2 + \tau_{11.top.weld}^2)}}{\frac{f_y}{(\gamma_{M0})}} = 31.4\%$$

Utilization ratio top weld

$$\eta_2 = 12.6\%$$

Utilization of Local Web Capacity

$$\left(\frac{\sigma_{x.Ed}}{\frac{f_y}{\gamma_{M0}}} \right)^2 + \left(\frac{\sigma_{z.Ed}}{\frac{f_y}{\gamma_{M0}}} \right)^2 - \left(\frac{\sigma_{x.Ed}}{\frac{f_y}{\gamma_{M0}}} \right) \cdot \left(\frac{\sigma_{z.Ed}}{\frac{f_y}{\gamma_{M0}}} \right) + 3 \cdot \left(\frac{\tau_{Ed}}{\frac{f_y}{\gamma_{M0}}} \right)^2 = 88.59\%$$

Utilization Ratio Von Misses

Fatigue and FLS

Safety Factors

$$\gamma_{Mf} := 1.35$$

$$\gamma_{Ff} := 1$$

$$v_s := 140 \frac{km}{hr} = 38.889 \frac{m}{s}$$

$$L_{\phi.MG} := \frac{L}{m} = 10.6$$

Train Speed in m/s

Length of bridge (unitless)

Fatigue Load Constant

Equations are from EN 1991-2 Annex D

$$K_{dyn} := \frac{v_s}{160 \frac{m}{s}} = 0.243$$

D.3 for $L \leq 20 \text{ m}$

$$\varphi_{Fat.MG} := \frac{K_{dyn}}{1 - K_{dyn} + K_{dyn}^4} = 0.32$$

D.2

$$\varphi_{2.Fat.MG} := 0.56 \cdot e^{\frac{-L_{\phi.MG}^2}{100}} = 0.182$$

D.5

$$\phi_{Fat.MG} := 1 + \frac{1}{2} \cdot \left(\varphi_{Fat.MG} + \frac{1}{2} \cdot \varphi_{2.Fat.MG} \right) = 1.205$$

λ -Coefficient Method

Equations are from EN 1993-2 and "Fatigue design of steel and composite bridges" - Mohammad Al-Emrani and Mustafa Ayyül



Figure 2.11 Fatigue design procedure of railway bridges

$$\lambda_1 := 0.9 + \frac{0.93 - 0.9}{10 - 12.5} \cdot (L_{\phi.MG} - 12.5) = 0.923$$

Table 9.4, interpolation from the section on rail traffic with 25 t axles

$$\lambda_2 := 0.72$$

Table 9.5 and RKF, Chapter 1.3, load cycles for fatigue shall be limited to 2.5 million tonnes per year

$$\lambda_3 := 1.04$$

Table 9.6, Design life are set to 120 years

$$\lambda_4 := 1$$

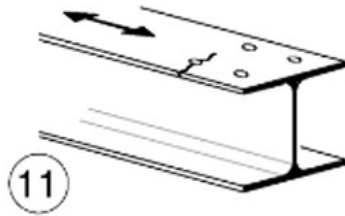
Table 9.7, The tension ratio becomes 1 when there is only one track

$$\lambda_{max} := 1.4$$

eq. 9.15

$$\lambda := \min(\lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4, \lambda_{max}) = 0.691$$

Detail 11, Hole in the Upper Flange for Attaching Sleeper



$$\Delta M_{Fat.LM71.11} := \frac{M_{max.train.K}}{2} \cdot \phi_{Fat.MG} \cdot 0.6 = 732.888 \text{ kN} \cdot \text{m} \quad \text{Maximum moment for fatigue in L/2}$$

$$\sigma_{max.1} := \frac{\Delta M_{Fat.LM71.11}}{I_y} \cdot (Z_{tp}) = 69.95 \text{ MPa} \quad \text{Maximum, top of the I-girder}$$

$$\sigma_{min.1} := 0$$

$$\Delta \sigma_{Ed.top.flange} := |\sigma_{max.1} - \sigma_{min.1}| = 69.95 \text{ MPa}$$

$$\Delta \sigma_{E.1} := \lambda \cdot \Delta \sigma_{Ed.top.flange} = 48.335 \text{ MPa}$$

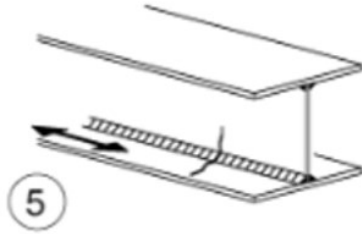
$$\Delta \sigma_{C.1} := 90 \cdot \text{MPa}$$

if $\Delta \sigma_{E.1} < \frac{\Delta \sigma_{C.1}}{\gamma_{Mf}}$ "Fatigue is OK" else "Fatigue is NOT OK"	= "Fatigue is OK"
--	-------------------

Check according to "Fatigue design of steel and composite bridges" -
Mohammad Al-Emrani and Mustafa Ayg  l

$$\frac{\Delta \sigma_{E.1}}{\frac{\Delta \sigma_{C.1}}{\gamma_{Mf}}} = 72.502\%$$

Detail 5, Main Girder Fillet Weld



$$\Delta M_{Fat.LM71.5} := \frac{M_{max.train.K}}{2} \cdot \phi_{Fat.MG} = (1.221 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$\sigma_{max.2} := \frac{\Delta M_{Fat.LM71.5}}{I_y} \cdot (-Z_{tp} + t_{fu}) = -112.056 \text{ MPa}$$

$$\sigma_{min.2} := 0$$

$$\Delta \sigma_{Ed.MG.fw} := |\sigma_{max.2} - \sigma_{min.2}| = 112.056 \text{ MPa}$$

$$\Delta \sigma_{E.2} := \lambda \cdot \Delta \sigma_{Ed.MG.fw} = 77.43 \text{ MPa}$$

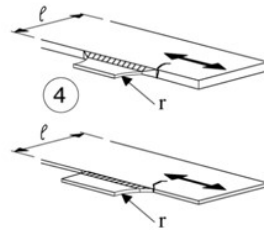
$$\Delta \sigma_{C.2} := 100 \cdot \text{MPa}$$

if $\Delta \sigma_{E.2} < \frac{\Delta \sigma_{C.2}}{\gamma_{Mf}}$ "Fatigue is OK" else "Fatigue is NOT OK"	= "Fatigue is NOT OK"
--	-----------------------

$$\frac{\Delta \sigma_{E.2}}{\Delta \sigma_{C.2}} = 104.53\%$$

Detail 4, Attachment For Truss-System

90	$\frac{r}{L} \geq \frac{1}{3}$ eller $r > 150\text{mm}$
71	$\frac{1}{6} \leq \frac{r}{L} \leq \frac{1}{3}$
50	$\frac{r}{L} < \frac{1}{6}$



L: anslutningslängd som vid detalj 1, 2 eller 3

$$M_{max.train.K.x1} := 2055.31 \text{ kN} \cdot \text{m}$$

$$x1 = 5\text{m}$$

$$\Delta M_{Fat.LM71.4} := \frac{M_{max.train.K.x1}}{2} \cdot \phi_{Fat.MG} = (1.239 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$\sigma_{max.3} := \frac{\Delta M_{Fat.LM71.4}}{I_y} \cdot (Z_{tp}) = 118.223 \text{ MPa}$$

$$\sigma_{min.3} := 0$$

$$\Delta \sigma_{Ed.attach.ts} := |\sigma_{max.3} - \sigma_{min.3}| = 118.223 \text{ MPa}$$

$$\Delta \sigma_{E.3} := \lambda \cdot \Delta \sigma_{Ed.attach.ts} = 81.691 \text{ MPa}$$

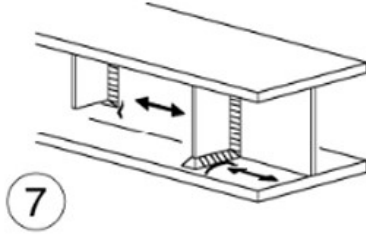
$$\Delta \sigma_{C.3} := 71 \text{ MPa}$$

if $\Delta \sigma_{E.3} < \frac{\Delta \sigma_{C.3}}{\gamma_{Mf}}$ "Fatigue is OK" else "Fatigue is NOT OK"	= "Fatigue is NOT OK"
--	-----------------------

$$\frac{\Delta \sigma_{E.3}}{\Delta \sigma_{C.3}} = 155.33\%$$

$$\gamma_{Mf}$$

Detail 7, Welding of Flange Stiffeners (Normal Stress)



$$M_{max.train.K.x2} := 1989.92 \text{ kN} \cdot \text{m}$$

$$x2 = 4.24 \text{ m}$$

$$\Delta M_{Fat.LM71.7} := \frac{M_{max.train.K.x2}}{2} \cdot \phi_{Fat.MG} = (1.199 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$\sigma_{max.4} := \frac{\Delta M_{Fat.LM71.7}}{I_y} \cdot (-Z_{tp} + t_{fu}) = -110.017 \text{ MPa}$$

$$\sigma_{min.4} := 0$$

$$\Delta \sigma_{Ed.stiff} := |\sigma_{max.4} - \sigma_{min.4}| = 110.017 \text{ MPa}$$

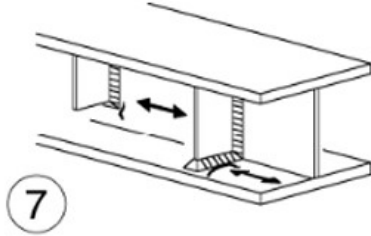
$$\Delta \sigma_{E.4} := \lambda \cdot \Delta \sigma_{Ed.stiff} = 76.021 \text{ MPa}$$

$$\Delta \sigma_{C.4} := 80 \text{ MPa}$$

$\text{if } \Delta \sigma_{E.4} < \frac{\Delta \sigma_{C.4}}{\gamma_{Mf}}$	= "Fatigue is NOT OK"
$\parallel \text{"Fatigue is OK"}$	
else	
$\parallel \text{"Fatigue is NOT OK"}$	

$$\frac{\Delta \sigma_{E.4}}{\frac{\Delta \sigma_{C.4}}{\gamma_{Mf}}} = 128.28\%$$

Detail 7, Welding of Web and Stiffeners (Principal Stresses)



$$V_{max.train.K.x2} := 369.77 \text{ kN} \quad V_{min.train.K.x2} := -183.96 \cdot \text{kN} \quad x2 = 4.24\text{m}$$

$$\Delta V_{ed.support.7} := \frac{(V_{max.train.K.x2} - V_{min.train.K.x2})}{2} \cdot \phi_{Fat.MG} = 333.713 \text{ kN}$$

$$\sigma_{max.5} := \frac{\Delta M_{Fat.LM71.7}}{I_y} \cdot (-Z_{tp} + t_{fu} + r_{notch}) = -98.904 \text{ MPa}$$

$$\Delta \sigma_{Ed.stiff.2} := |\sigma_{max.5}| = 98.904 \text{ MPa}$$

$$\Delta \tau_{11.stiffener} := \frac{\Delta V_{ed.support.7} \cdot S_{ztp.bot.sym}}{I_y \cdot t_w} = 17.074 \text{ MPa}$$

$$\Delta \sigma_{Ed.princ} := \frac{\Delta \sigma_{Ed.stiff.2}}{2} + \sqrt{\left(\frac{\Delta \sigma_{Ed.stiff.2}}{2}\right)^2 + \Delta \tau_{11.stiffener}^2} = 101.768 \text{ MPa}$$

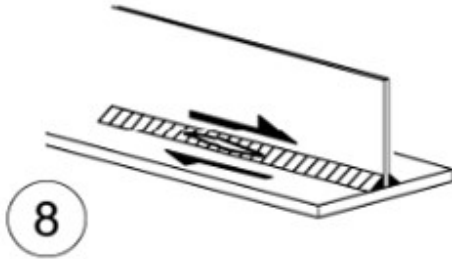
$$\Delta \sigma_{E.5} := \lambda \cdot \Delta \sigma_{Ed.princ} = 70.321 \text{ MPa}$$

$$\Delta \sigma_{C.5} := 80 \text{ MPa}$$

if $\Delta \sigma_{E.5} < \frac{\Delta \sigma_{C.5}}{\gamma_{Mf}}$ "Fatigue is OK" else "Fatigue is NOT OK"	= "Fatigue is NOT OK"
--	-----------------------

$$\frac{\Delta \sigma_{E.5}}{\frac{\Delta \sigma_{C.5}}{\gamma_{Mf}}} = 118.67\%$$

Detail 8, Longitudinal Welding Between Web and Flange



$$V_{max.train.K} = 854.75 \text{ kN}$$

$$\Delta V_{ed.support.8} := \frac{V_{max.train.K}}{2} \cdot \phi_{Fat.MG} = 515.127 \text{ kN}$$

$$\Delta \tau_{11.bottom.weld} := \frac{\Delta V_{ed.support.8} \cdot S_{ztp.bot.sym}}{I_y \cdot b_{bot}} = 49.416 \text{ MPa}$$

$$\Delta \tau_{E.6} := \lambda \cdot \Delta \tau_{11.bottom.weld} = 34.146 \text{ MPa}$$

$$\Delta \tau_{C.6} := 80 \text{ MPa}$$

if $\Delta \tau_{E.6} < \frac{\Delta \tau_{C.6}}{\gamma_{Mf}}$ "Fatigue is OK" else "Fatigue is NOT OK"	= "Fatigue is OK"
--	-------------------

$$\frac{\Delta \tau_{E.6}}{\frac{\Delta \tau_{C.6}}{\gamma_{Mf}}} = 57.62\%$$

B

Appendix B

The primary functions that have been developed for this model are:

- **Function `Generate_Influence_Lines`:** This function generates influence line matrices for bending moments, shear forces, and vertical deflections. It is based on a 1D Finite Element Method (FEM) using Euler-Bernoulli beam theory, where the continuous beam is discretized into elements with two degrees of freedom per node and variable flexural stiffness (EI). By applying a moving unit load and solving the global stiffness matrix, it outputs the structural response matrices required to evaluate the different load effects vertical loads.
- **Function `Load_Traffic_IL`:** This function applies the train load model (e.g., LM71) across the generated influence lines to identify the most adverse load positions along the bridge. It evaluates both discrete axle loads and distributed loads in accordance with *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003), yielding the maximum and minimum values for bending moment and shear force along a beam.
- **Function `calc_selfweight_IL`:** This function calculates the bending moments and shear forces, resulting from the permanent dead loads of the bridge. It calculates a total distributed line load by combining the self-weight of the main girders, based on their varying cross-sectional areas, with additional superimposed dead loads. The static structural response at each node is then obtained by numerically integrating this total distributed load over the influence line matrices.
- **Function `calc_wind_load_IL`:** This function calculates the horizontal and vertical load effects induced by wind pressure. It determines the total horizontal line load based on the projected exposed areas of the bridge and the train. Furthermore, it calculates the overturning moment about the section's centre of gravity. This moment is structurally resolved into a vertical force couple acting on the main girders. By numerically integrating this vertical line load over the influence line matrices, the function outputs the resulting vertical bending moments and shear forces in the main girder.
- **Function `calc_traction_load`:** This function evaluates the structural effects of longitudinal traction and braking forces from train traffic. It determines the governing force magnitude based on the characteristic limits defined in *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003). By evaluating the accumulation of these

distributed forces towards the fixed bearing, the function generates symmetric worst-case curves for axial compression, as well as the associated vertical bending moments and shear forces induced by the load's eccentricity relative to the section's centre of gravity.

- **Function `calc_nosing_load`:** This function calculates the structural effects of transverse nosing forces (lateral impacts from trains). It determines the maximum local lateral bending moment in one beam by modelling it as a simply supported beam spanning between the horizontal cross-girders, yielding a constant envelope for lateral bending along the bridge.
- **Function `Calfem_truss_beam`:** This function performs a 2D hybrid Finite Element Analysis (FEA) using the CALFEM toolbox to evaluate the structural response of the horizontal bracing system under wind and transverse nosing loads. The main girders are modelled as beam elements with flexural stiffness, while the cross-beams and diagonals are modelled as pinned truss elements. By assembling and solving the global stiffness matrix for the specified discrete load cases, the function extracts the governing maximum axial forces (tension and compression) for the main girders and the individual truss members.
- **Function `URm_LT`:** This ULS function evaluates the structural resistance of the main girders under combined bending and axial forces. It computes effective cross-sectional properties by iteratively applying reduction factors for shear lag and, if needed, local plate buckling (Class 4 cross-sections). The function outputs utilization ratios for three primary checks: verification of yield stress under combined bending and axial force, global out-of-plane stability evaluating combined flexural and lateral-torsional buckling using the General Method (treating the two main girders as an interacting system), and local lateral-torsional buckling of the compressed flange between transverse bracing using the Simplified Method. In addition to this flange induced buckling is considered only utilizing elastic moment resistance.
- **Function `URv`:** This ULS function evaluates the shear capacity of the main girder webs, considering effects from shear buckling of the web. It determines the design shear resistance by applying the appropriate shear buckling reduction factors based on the web slenderness and the spacing of transverse stiffeners. Furthermore, the function evaluates the transverse stiffeners. It verifies their minimum required stiffness to act as rigid boundaries for the web panels. If the stiffeners attract compressive forces due to tension field action in the post-buckling regime, their structural stability is assessed by modelling them as effective columns, including part of the web panels, subject to combined axial compression and bending.
- **Function `UR_MV_Interaction`:** This ULS function evaluates the cross-sectional resistance of the main girders under the combined effects of bending moments, shear forces, and axial forces. In accordance with Eurocode provisions, the interaction verification is only triggered for sections where the applied design shear force exceeds 50% of the web's shear capacity. When activated, the function calculates a reduced bending capacity to account for the simultaneous presence of shear and axial stresses, applying the appropriate interaction criteria dependent on the slenderness of the web.

- **Function UR_weld_longitudinal:** This ULS function evaluates the structural resistance of the longitudinal flange-to-web connections. It adapts the assessment criteria depending on whether fillet or full-penetration butt welds are used. At the bottom flange, the evaluation is governed by longitudinal shear flow. At the top flange, the function additionally incorporates the effects of vertical concentrated patch loads from the train axles. For full-penetration butt welds, the resistance is dictated by the web plate. The function evaluates shear yielding, web crippling (from patch load), the interaction between local vertical forces and global bending, and local Von Mises yielding under combined multi-axial stresses. Conversely, for fillet welds, the corresponding multi-axial stress states are evaluated directly across the effective weld throat thickness utilizing the directional method.
- **Function UR_truss_buckling:** This ULS function evaluates the structural resistance of the horizontal bracing system, comprising the UPE cross-beams and L-profile diagonals, subjected to transverse wind and train nosing loads. Assuming idealized pinned connections, the members are evaluated exclusively for axial forces. The function verifies the tension capacity against gross section yielding and assesses the compression capacity against flexural buckling. To accurately determine the buckling resistance, the effective buckling lengths for the respective members are derived from the transverse distance between the main girder webs and the geometric boundaries of the connecting stiffeners.
- **Function calc_SLS_target:** This SLS function evaluates the vertical deflection at a specific target point along the bridge, typically the mid-span. Utilizing the influence lines generated from the 1D Finite Element model, which accounts for the varying flexural stiffness along the girders, it computes the static deflections from permanent load, vertical wind pressures and the LM71. Furthermore, the function approximates the eigenfrequency of the bridge based on the mid-span deflection under permanent load. This frequency is verified against the upper and lower limits according to *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003) to prevent dynamic resonance effects.
- **Function UR_FLS_HFMI:** This FLS function evaluates the fatigue resistance of the specified structural details using the simplified λ -coefficient method. It encompasses both standard as-welded and HFMI-treated details, in accordance with the upcoming revision of *Eurocode 3: Design of steel structures - part 1-9: Fatigue (New)* (Swedish Standards Institute, 2022) and its Annex F. For HFMI-treated details, the function scales the reference fatigue strength based on the material yield strength according to Equation 2.6. It explicitly accounts for mean stress effects using the penalty factor λ_{HFMI} according to Equation 2.17. Furthermore, the function incorporates an overload check in accordance with Section 2.2.2.2.
- **Function UR_FLS_damage_HFMI_acc:** This FLS function evaluates the accumulated fatigue damage using the ESR method. It encompasses both standard as-welded and HFMI-treated details, in accordance with the upcoming revision of *Eurocode 3: Design of steel structures - part 1-9: Fatigue (New)* (Swedish Standards Institute, 2022) and its Annex F. The function generates a

stress history from the traffic mix (according to *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges* (Swedish Standards Institute, 2003)), which is subsequently processed to extract stress cycles using the rainflow counting method. These cycles are then used to iteratively compute the ESR and the corresponding number of cycles according to Equation 2.13 and 2.14, from which the accumulated fatigue damage (D) is determined. For HFMI-treated details, the function scales the reference fatigue strength based on the material yield strength according to Equation 2.6. It accounts for mean stress effects using the penalty factor λ_{HFMI} according to Equation 2.17. Furthermore, the function considers the overload effects in accordance with Section 2.2.2.2, reverting to the as-welded capacity if limit values is surpassed.

C

Appendix C

This appendix presents a comparison between manual calculations and examples from the book *Fatigue Design of Steel and Composite Bridges* by (Al-Emrani & Aygöl, 2014), in order to highlight the inaccuracies in the results when using the ESR method.

Verification calulation of damage on details from example								
Delta sigma abow knee point								
Detail 1			Detail 2			Detail 3		
		Contribution to equivalent stress for each stressrange			Contribution to equivalent stress for each stressrange			Contribution to equivalent stress for each stressrange
Antal Cykler	Stress range		Antal Cykler	Stress range		Antal Cykler	Stress range	
16	6	0	16	6	0	71	6	0
6	7	0	6	7	0	6	7	0
78	8	0	78	8	0	29	9	0
52	9	0	52	10	0	84	10	0
13	10	0	13	11	0	65	11	0
26	12	0	26	13	0	13	13	0
144	22	0	144	22	493959,9751	144	21	0
26	23	0	26	24	137798,7952	26	22	0
13	24	54269,04113	13	25	84500,49626	13	23	0
39	30	496847,9107	39	30	630792,8245	39	27	293383,7228
16	39	756825,2361	16	37	738489,7415	16	32	281463,7448
13	43	1001931,931	13	42	963144	13	36	412105,5311
16	45	1458000	16	43	1272112	16	37	581676,0605
6	47	622938	6	46	584016	6	38	249242,3405
Delta sigma below knee point								
Detalj 1			Detail 2			Detail 3		
		Contribution to equivalent stress for each stressrange			Contribution to equivalent stress for each stressrange			Contribution to equivalent stress for each stressrange
Antal Cykler	Stress range		Antal Cykler	Stress range		Antal Cykler	Stress range	
16	6	0	16	6	0	71	6	0
6	7	0	6	7	0	6	7	0
78	8	0	78	8	0	29	9	0
52	9	0	52	10	0	84	10	0
13	10	0	13	11	0	65	11	0
26	12	0	26	13	0	13	13	0
144	22	0	144	22	742123008	144	21	0
26	23	0	26	24	207028224	26	22	0
13	24	103514112	13	25	126953125	13	23	0
39	30	947700000	39	30	947700000	39	27	559607373
16	39	1443587184	16	37	1109503312	16	32	536870912
13	43	1911109759	13	42	1447022752	13	36	786060288
16	45	2781025280	16	43	1911214737	16	37	1109503312
6	47	1188207357	6	46	877422731,6	6	38	475411008
Damages for details dependent on abow or bellow knee point								
	Detail 1	Detail 2	Detail 3					
D_tot_hand_calc [-]	0,46	0,74	0,19					
D_tot_PDM [-]	0,43	0,73	0,17					
Difference [%]	6,33%	1,51%	11,07%					

m_1		3	
m_2		5	
Delta sigma abow knee point			
	Detail 1	Detail 2	Detail 3
Total years	120	120	120
Total days	43800	43800	43800
Detalj klass	80	71	80
gamma_MF	1,35	1,35	1,35
gamma_Ff	1	1	1
Delta sigma_D	58,96	52,327	58,96
Delta sigma_L	32,36904	28,727523	32,36904
Delta sigma_D_DIM	43,67407407	38,76074074	43,67407407
Delta sigma_L_DIM	23,97706667	21,27964667	23,97706667
Delta_sigma_equivalent	21,15155453	21,94664721	14,97805073
N_equivalent	44016372,64	27544929,25	123958191,5
n_tot	464	464	541
D_day	1,05415E-05	1,68452E-05	4,36437E-06
D_tot	0,461719101	0,737820011	0,191159614
m_1		3	
m_2		5	
Delta sigma below knee point			
	Detail 1	Detail 2	Detail 3
Total years	120	120	120
Total days	43800	43800	43800
Detalj klass	80	71	80
gamma_MF	1,35	1,35	1,35
gamma_Ff	1	1	1
Delta sigma_D	58,96	52,327	58,96
Delta sigma_L	32,36904	28,727523	32,36904
Delta sigma_D_DIM	43,67407407	38,76074074	43,67407407
Delta sigma_L_DIM	23,97706667	21,27964667	23,97706667
Delta_sigma_equivalent	28,26798533	27,55356113	22,98066865
N_equivalent	44016372,64	27544929,25	123958191,5
n_tot	464	464	541
D_day	1,05415E-05	1,68452E-05	4,36437E-06
D_tot	0,461719101	0,737820011	0,191159614
Damages for details dependent on abow or bellow knee point			
	Detail 1	Detail 2	Detail 3
D_tot_hand_calc [-]	0,46	0,74	0,19
D_tot_PDM [-]	0,43	0,73	0,17
Difference [%]	6,33%	1,51%	11,07%

Savings from girder height					
Material			Total		
Standard	our	Difference	Standard	our	Difference
313	217	-44,24%	777	600	-29,50%
626	492	-27,24%	1550	*****	-22,05%
1000	811	-23,30%	2390	*****	-15,18%

D

Appendix D

CASE	Total [SEK]	UP_LocalBending	UP_LocalID	UP_Shear	UP_Stiffener	UP_Interaction	UP_WebTop	UP_WebFoot	UP_Ung	UP_L	UP_Direction	"Range induced bending"	"UP_Eigenfreq"	MAX x1.5 CSM method
AW S355, L= 10 m	257682,1263	67,26%	27,71%	59,82%	0,00%	66,72%	63,53%	60,13%	31,93%	63,54%	62,82%	20,44%	105,50%	98,10%
AW S355, L= 20 m	605871,665	99,15%	43,14%	83,08%	0,00%	99,01%	86,38%	54,16%	54,55%	88,08%	88,04%	43,09%	92,80%	99,92%
AW S355, L= 30 m	1270261,432	81,10%	25,21%	61,25%	0,00%	87,06%	67,64%	47,55%	44,47%	88,49%	90,67%	42,11%	85,69%	99,50%
AW S355, L= 40 m	2074660,383	87,51%	24,20%	64,15%	0,00%	96,87%	74,43%	45,82%	35,13%	99,02%	99,96%	59,13%	77,06%	99,52%
HMFt-treated S355, L= 10 m	204969,9071	91,88%	61,48%	99,11%	0,00%	84,18%	92,54%	92,28%	39,34%	93,57%	90,53%	28,37%	97,86%	88,12%
HMFt-treated S355, L= 20 m	586513,4681	99,88%	62,61%	83,79%	0,00%	99,94%	84,60%	52,71%	29,25%	85,65%	99,61%	37,71%	90,20%	51,27%
HMFt-treated S355, L= 30 m	1208432,009	99,54%	55,86%	62,06%	46,26%	98,86%	85,13%	44,80%	44,68%	88,79%	99,61%	73,05%	86,29%	90,90%
HMFt-treated S355, L= 40 m	2218018,331	91,67%	33,43%	60,73%	16,70%	93,91%	80,80%	40,55%	10,95%	67,84%	97,67%	83,18%	82,26%	62,03%
HMFt-treated S460, L= 10 m	225140,5591	71,58%	54,27%	87,07%	0,00%	65,29%	71,42%	71,22%	35,42%	88,05%	90,53%	36,42%	97,86%	88,12%
HMFt-treated S460, L= 20 m	650074,5274	95,85%	95,34%	86,63%	93,86%	99,46%	70,88%	53,18%	27,27%	95,58%	100,07%	94,04%	90,77%	79,00%
HMFt-treated S460, L= 30 m	1316468,75	83,17%	41,90%	50,24%	0,00%	89,76%	71,72%	44,19%	25,59%	82,59%	99,60%	73,62%	85,61%	88,31%
HMFt-treated S460, L= 40 m	2340846,695	83,48%	40,04%	50,09%	15,33%	85,81%	72,22%	31,32%	21,32%	100,06%	99,94%	89,09%	82,49%	79,14%
HMFt-treated S690, L= 10 m	241676,8752	47,34%	44,14%	71,09%	0,00%	42,97%	47,61%	47,48%	31,08%	95,28%	90,53%	55,05%	97,86%	88,00%
HMFt-treated S690, L= 20 m	700260,7546	60,77%	44,61%	53,84%	1,58%	61,25%	54,44%	38,52%	16,06%	90,58%	100,02%	84,69%	89,76%	55,86%
HMFt-treated S690, L= 30 m	1500973,445	58,54%	30,36%	38,83%	0,00%	0,00%	54,62%	34,52%	11,88%	64,88%	97,93%	88,30%	85,17%	54,36%
HMFt-treated S690, L= 40 m	2615869,545	77,76%	51,89%	38,37%	0,00%	0,00%	50,10%	22,89%	14,05%	82,20%	98,12%	94,52%	80,80%	61,31%

E

Appendix E

This appendix includes all the Excel calculations used to derive the results for the ESR method presented in the report.

Combinations							
welde, 1=HFMI)	welde d or	("Factory" if in factory or	Length	Steel	Included In Min data (=1)	Sorting away weld with 3 mm throat	number of bridges used
0,00	As-weld	factory	10000,00	355,00	1,00	3,00	3,00
0,00	As-weld	factory	20000,00	355,00	1,00	3,00	4,00
0,00	As-weld	factory	30000,00	355,00	1,00	3,00	7,00
0,00	As-weld	factory	40000,00	355,00	1,00	3,00	5,00
1,00	HFMI-t	factory	10000,00	355,00	1,00	3,00	3,00
1,00	HFMI-t	factory	20000,00	355,00	1,00	3,00	6,00
1,00	HFMI-t	factory	30000,00	355,00	1,00	3,00	3,00
1,00	HFMI-t	factory	40000,00	355,00	1,00	3,00	2,00
1,00	HFMI-t	factory	10000,00	460,00	1,00	3,00	3,00
1,00	HFMI-t	factory	20000,00	460,00	1,00	3,00	4,00
1,00	HFMI-t	factory	30000,00	460,00	1,00	3,00	3,00
1,00	HFMI-t	factory	40000,00	460,00	1,00	3,00	3,00
1,00	HFMI-t	factory	10000,00	690,00	1,00	3,00	3,00
1,00	HFMI-t	factory	20000,00	690,00	1,00	3,00	3,00
1,00	HFMI-t	factory	30000,00	690,00	1,00	3,00	1,00
1,00	HFMI-t	factory	40000,00	690,00	1,00	3,00	1,00

Train mix	Plot Name	Total [SEK]	Material [SEK]
Standard train mix	As-welded 355 Standard t	257682,13	110359,82
Standard train mix	As-welded 355 Standard t	605871,67	217812,41
Standard train mix	As-welded 355 Standard t	1270261,43	492066,55
Standard train mix	As-welded 355 Standard t	2074660,38	811325,09
Standard train mix	HFMI-treated factory 355	204969,91	76138,55
Standard train mix	HFMI-treated factory 355	586513,47	199583,04
Standard train mix	HFMI-treated factory 355	1208432,01	415624,15
Standard train mix	HFMI-treated factory 355	2218018,33	723523,48
Standard train mix	HFMI-treated factory 460	225140,56	96309,20
Standard train mix	HFMI-treated factory 460	650074,53	242556,52
Standard train mix	HFMI-treated factory 460	1316468,75	518711,17
Standard train mix	HFMI-treated factory 460	2340846,69	864301,54
Standard train mix	HFMI-treated factory 690	241676,88	112845,51
Standard train mix	HFMI-treated factory 690	700260,75	291559,70
Standard train mix	HFMI-treated factory 690	1500973,45	645614,98
Standard train mix	HFMI-treated factory 690	2615869,55	1052679,50

Category COST			
painting [SEK]	Welding [SEK]	HFMI [SEK]	Total Other Cost [SEK]
58077,50	41255,40	0,00	47989,40
179444,08	105657,82	0,00	102957,35
405240,00	185726,68	0,00	187228,20
629448,60	333899,62	0,00	299987,08
46575,00	37120,68	3470,72	41664,97
169499,08	100993,16	16174,00	100264,19
396926,50	173399,34	31483,32	190998,71
793154,60	309470,85	57881,74	333987,67
46575,00	37120,68	3470,72	41664,97
174847,84	100197,38	15944,37	116528,42
404409,50	172081,89	31145,97	190120,22
770752,20	314050,94	57418,72	334323,29
46575,00	37120,68	3470,72	41664,97
190023,92	102026,87	17228,68	99421,58
447917,75	178737,61	33173,80	195529,31
813074,28	322221,28	57750,09	370144,39

Minimum values Cost & Weight

Relative change in cost compared to S355 As-welded				
Change in total cost	Change in material cost	Change in painting cost	Change in welding cost	Change in "other costs"
0,00%	0,00%	0,00%	0,00%	0,00%
0,00%	0,00%	0,00%	0,00%	0,00%
0,00%	0,00%	0,00%	0,00%	0,00%
0,00%	0,00%	0,00%	0,00%	0,00%
20,46%	31,01%	19,81%	10,02%	13,18%
3,20%	8,37%	5,54%	4,41%	2,62%
4,87%	15,53%	2,05%	6,64%	-2,01%
-6,91%	10,82%	-26,01%	7,32%	-11,33%
12,63%	12,73%	19,81%	10,02%	13,18%
-7,30%	-11,36%	2,56%	5,17%	-13,18%
-3,64%	-5,41%	0,20%	7,35%	-1,54%
-12,83%	-6,53%	-22,45%	5,94%	-11,45%
6,21%	-2,25%	19,81%	10,02%	13,18%
-15,58%	-33,86%	-5,90%	3,44%	3,43%
-18,16%	-31,20%	-10,53%	3,76%	-4,43%
-26,09%	-29,75%	-29,17%	3,50%	-23,39%

Chategory WEIGHT			Relative change in weight compared to S355 As-welded		
Weight plates [kg]	Weight profiles [kg]	Weight Total [kg]	Change in weight of hot-rolled plates	Weight profiles [%]	Change in total weight
9462	222	9684	0,00%	0,00%	0,00%
18645	535	19180	0,00%	0,00%	0,00%
43200	674	43874	0,00%	0,00%	0,00%
72214	1077	73292	0,00%	0,00%	0,00%
6381	256	6638	32,56%	-15,61%	31,46%
16976	572	17548	8,95%	-6,93%	8,51%
36262	861	37122	16,06%	-27,72%	15,39%
64266	1447	65714	11,01%	-34,31%	10,34%
6381	256	6638	32,56%	-15,61%	31,46%
16286	518	16804	12,65%	3,06%	12,38%
36035	908	36944	16,59%	-34,81%	15,80%
60894	1006	61900	15,68%	6,66%	15,54%
6381	230	6612	32,56%	-3,89%	31,73%
16811	516	17326	9,84%	3,53%	9,66%
38089	1032	39121	11,83%	-53,15%	10,83%
62934	959	63893	12,85%	10,97%	12,82%

CO ₂				
CO ₂ [kg]	Change in CO ₂ -equivalents	MAX ULS	MAX SLS (eigen frequency)	MAX SLS (deflection)
15107,27	0,00%	67,26%	105,50%	62,82%
29920,03	0,00%	99,15%	92,80%	88,04%
68443,45	0,00%	87,06%	85,69%	90,67%
114334,75	0,00%	96,87%	77,06%	99,96%
10354,60	31,46%	99,11%	97,86%	90,53%
27374,26	8,51%	99,94%	90,20%	99,61%
57911,01	15,39%	99,54%	86,29%	99,61%
102513,30	10,34%	93,91%	82,26%	97,67%
10752,86	28,82%	87,07%	97,86%	90,53%
27222,83	9,01%	99,46%	90,77%	100,07%
59848,48	12,56%	89,76%	85,61%	99,60%
100277,22	12,30%	89,09%	82,49%	99,94%
11371,89	24,73%	71,09%	97,86%	90,53%
29801,36	0,40%	84,69%	89,76%	100,02%
67287,36	1,69%	88,30%	85,17%	97,93%
109896,10	3,88%	94,52%	80,80%	98,12%

Category Utilization		
MAX FLS (ESR-method)	MAX FLS (Lambda-method)	Determining detail
98,10%	109,82%	WeldingBetweenFlange_Stif_ESR_D
99,92%	115,88%	WeldingBetweenFlange_Stif_ESR_D
99,50%	115,14%	AttachmentForL_Profile_ESR_D
99,52%	114,95%	WeldingBetweenFlange_Stif_ESR_D
88,12%	122,38%	MainGirderButtWeldbot/Top_ESR_D
51,27%	98,05%	MainGirderButtWeldbot/Top_ESR_D
90,90%	106,42%	AttachmentForL_Profile_ESR_D
62,03%	96,69%	AttachmentForL_Profile_ESR_D
88,12%	112,90%	MainGirderButtWeldbot/Top_ESR_D
79,00%	107,15%	WeldingBetweenFlange_Stif_ESR_D
88,31%	105,81%	AttachmentForL_Profile_ESR_D
79,14%	100,49%	AttachmentForL_Profile_ESR_D
88,00%	107,41%	MainGirderButtWeldbot/Top_ESR_D
55,86%	99,32%	MainGirderButtWeldbot/Top_ESR_D
54,36%	97,06%	MainGirderButtWeldbot/Top_ESR_D
61,31%	92,98%	WeldingBetweenFlange_Stif_ESR_D

Combinations						Plot Name	
Status (0=As-welde, 1=HFMI)	As-welde d or HFMI	HFMI location ("Factory" if in factory or aswelded, otherwise "On-site")	Length	Steel	Included In Mean data (=1)	Plot Name	
0,00	As-wel	factory	10000,00	355,00	1,00	As-welded 355	670
0,00	As-wel	factory	20000,00	355,00	1,00	As-welded 355	1330
0,00	As-wel	factory	30000,00	355,00	1,00	As-welded 355	1950
0,00	As-wel	factory	40000,00	355,00	1,00	As-welded 355	2390
1,00	HFMI-t	factory	10000,00	355,00	1,00	HFMI-treated factory 355	670
1,00	HFMI-t	factory	20000,00	355,00	1,00	HFMI-treated factory 355	1330
1,00	HFMI-t	factory	30000,00	355,00	1,00	HFMI-treated factory 355	2000
1,00	HFMI-t	factory	40000,00	355,00	1,00	HFMI-treated factory 355	2600
1,00	HFMI-t	factory	10000,00	460,00	1,00	HFMI-treated factory 460	670
1,00	HFMI-t	factory	20000,00	460,00	1,00	HFMI-treated factory 460	1330
1,00	HFMI-t	factory	30000,00	460,00	1,00	HFMI-treated factory 460	2000
1,00	HFMI-t	factory	40000,00	460,00	1,00	HFMI-treated factory 460	2670
1,00	HFMI-t	factory	10000,00	690,00	1,00	HFMI-treated factory 690	670
1,00	HFMI-t	factory	20000,00	690,00	1,00	HFMI-treated factory 690	1330
1,00	HFMI-t	factory	30000,00	690,00	1,00	HFMI-treated factory 690	2000
1,00	HFMI-t	factory	40000,00	690,00	1,00	HFMI-treated factory 690	2670

h_{tot} [mm]	n_{bays}	a_1 [mm]	a_{bw} [mm]	UpE_Profile
	4	2500	6	140
	10	2000	4	140
	14	2143	4	140
	20	2000	4	180
	4	2500	6	140
	9	2222	4	140
	14	2143	4	160
	19	2105	4	220
	4	2500	6	140
	9	2222	4	140
	14	2143	4	200
	19	2105	5	140
	4	2500	6	140
	9	2222	4	160
	14	2143	4	240
	19	2105	4	160

L_Profile	Number of splices [pcs]	x_Splice_1 [mm]	x_Splice_2 [mm]
808	0	No splice here!	No splice here!
909	2	4000	No splice here!
8010	2	10500	No splice here!
8010	4	10000	16500
8010	0	No splice here!	No splice here!
9011	2	4000	No splice here!
9011	2	9000	No splice here!
10012	4	9000	19500
8010	0	No splice here!	No splice here!
909	2	5000	No splice here!
909	2	9000	No splice here!
8012	4	9500	19000
808	0	No splice here!	No splice here!
8010	2	6000	No splice here!
909	2	10500	No splice here!
909	4	6500	18500

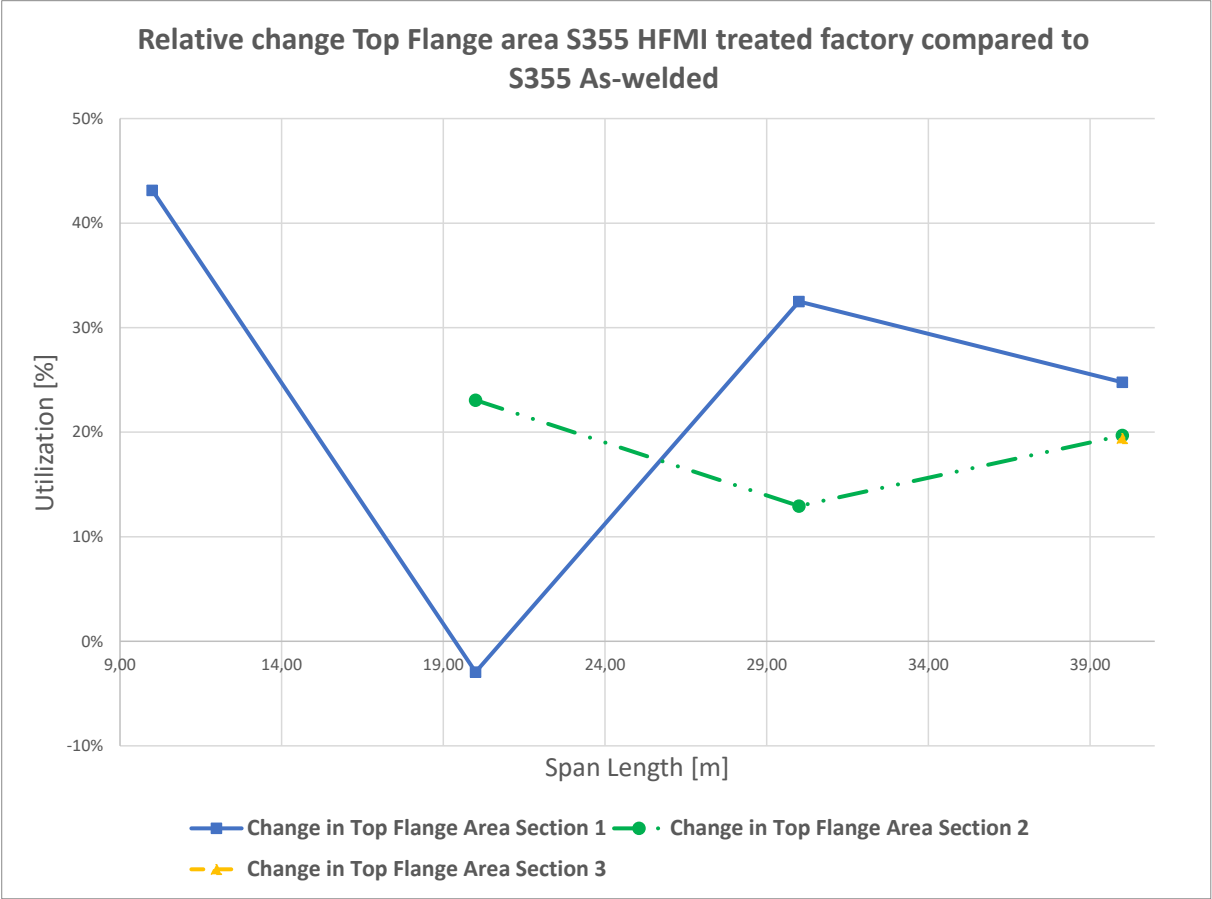
Minimum values L				
<i>x_Splice_3 [mm]</i>	<i>x_Splice_4 [mm]</i>	<i>tfo_1 [mm]</i>	<i>tfo_2 [mm]</i>	<i>tfo_3 [mm]</i>
No splice here!	No splice here!	60	60	60
No splice here!	16000	40	50	50
No splice here!	19500	50	45	45
23500	30000	50	60	60
No splice here!	No splice here!	60	60	60
No splice here!	16000	50	45	45
No splice here!	21000	60	45	45
20500	31000	45	50	60
No splice here!	No splice here!	60	60	60
No splice here!	15000	60	60	60
No splice here!	21000	40	40	40
21000	30500	45	50	45
No splice here!	No splice here!	60	60	60
No splice here!	14000	35	40	40
No splice here!	19500	35	50	50
21500	33500	35	50	60

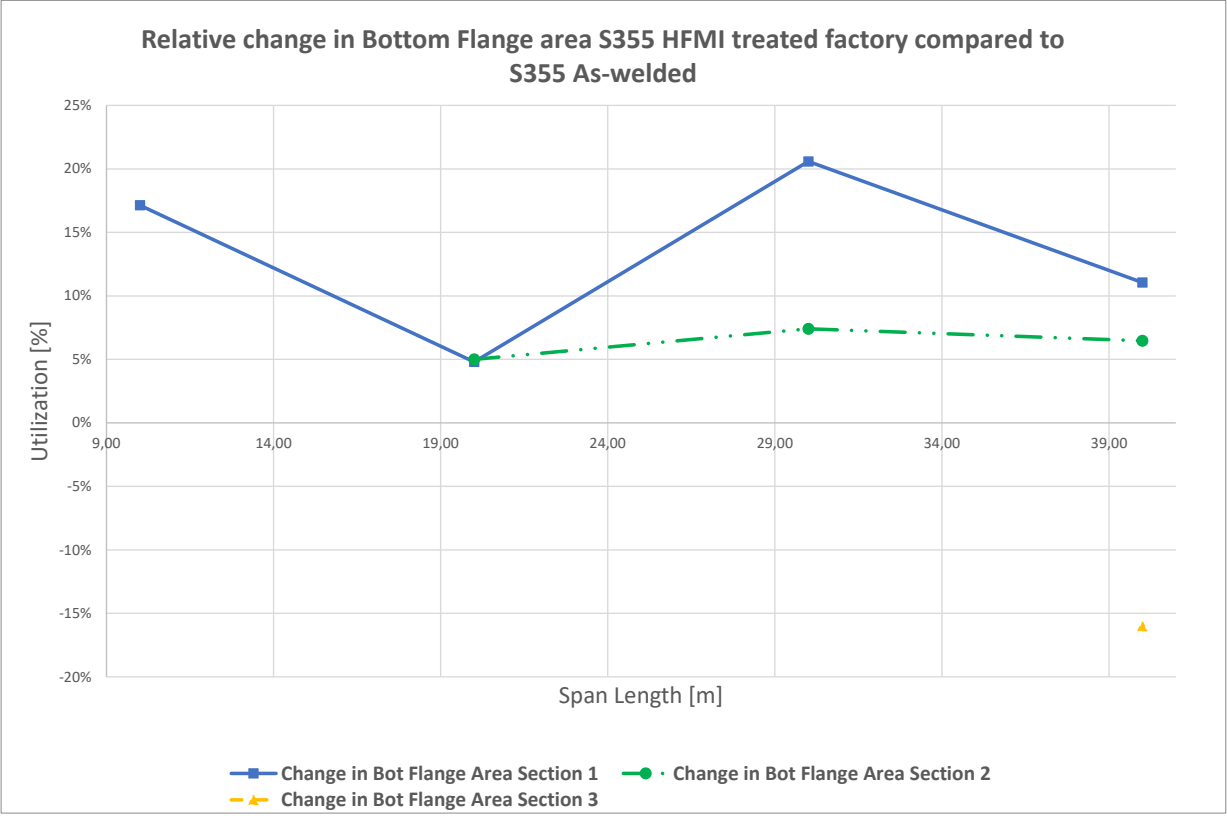
Dimensions						
Mean Dimensions						
Average top flange thickness [mm]	bfo_1 [mm]	bfo_2 [mm]	bfo_3 [mm]	Average top flange width [mm]	tw_1 [mm]	
60	510	510	510	510	15	
47	340	620	620	527	12	
47	640	850	850	780	15	
57	670	830	1030	843	15	
60	290	290	290	290	10	
47	280	530	530	447	12	
50	360	740	740	613	15	
52	560	800	830	730	15	
60	290	290	290	290	10	
60	250	420	420	363	12	
40	500	900	900	767	15	
47	480	670	1010	720	15	
60	290	290	290	290	10	
38	390	790	790	657	12	
45	530	590	590	570	15	
48	510	660	1230	800	15	

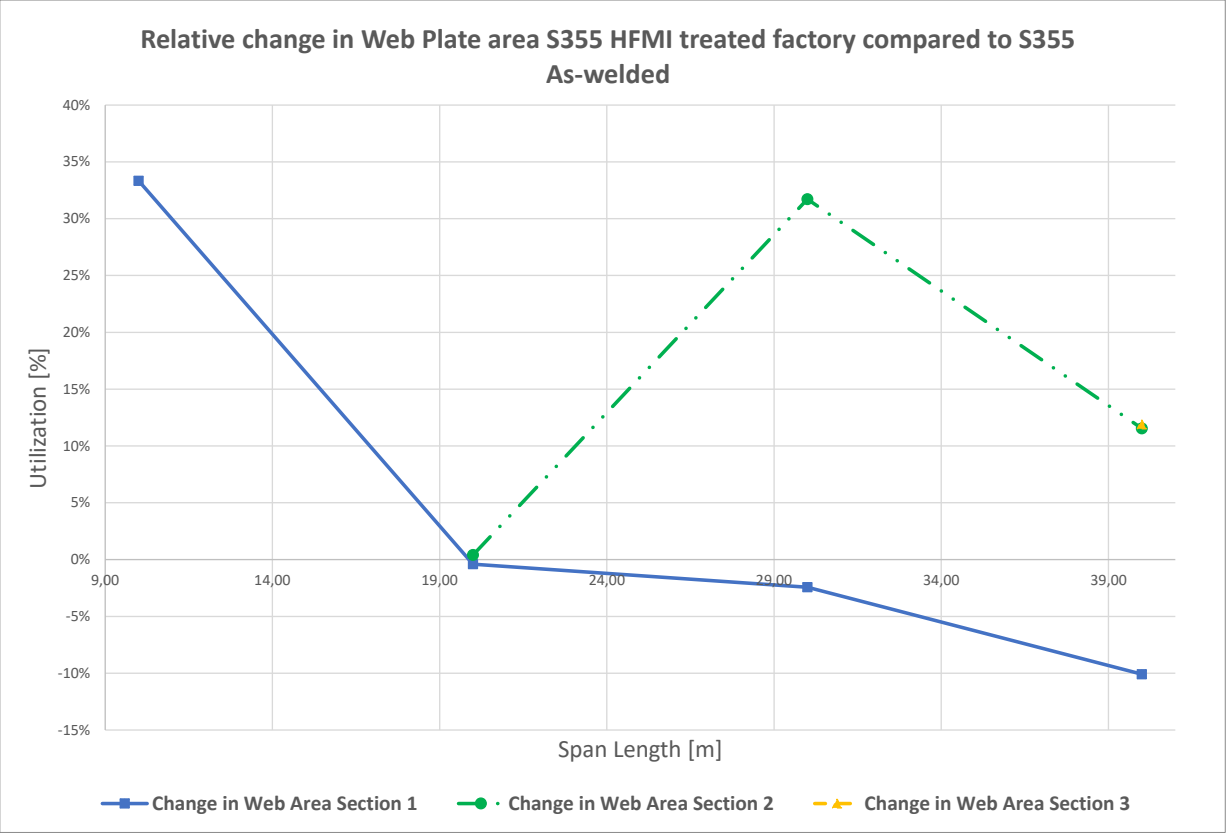
tw_2 [mm]	tw_3 [mm]	Avarage web thickness [mm]	hw_1 [mm]	hw_2 [mm]	
15	15	15	550	550	
12	12	12	1240	1230	
15	15	15	1850	1860	
15	15	15	2280	2270	
10	10	10	550	550	
12	12	12	1245	1225	
10	10	12	1895	1905	
12	12	13	2510	2510	
10	10	10	550	550	
8	8	9	1230	1210	
12	12	13	1915	1900	
12	20	16	2590	2570	
10	10	10	550	550	
12	12	12	1265	1245	
15	15	15	1930	1915	
15	25	18	2627	2560	

hw_3 [mm]	tfu_1 [mm]	tfu_2 [mm]	tfu_3 [mm]
550	60	60	60
1230	50	50	50
1860	50	45	45
2270	60	60	60
550	60	60	60
1225	35	60	60
1905	45	50	50
2500	45	40	40
550	60	60	60
1210	40	60	60
1900	45	60	60
2595	35	50	30
550	60	60	60
1245	30	45	45
1915	35	35	35
2570	8	60	40

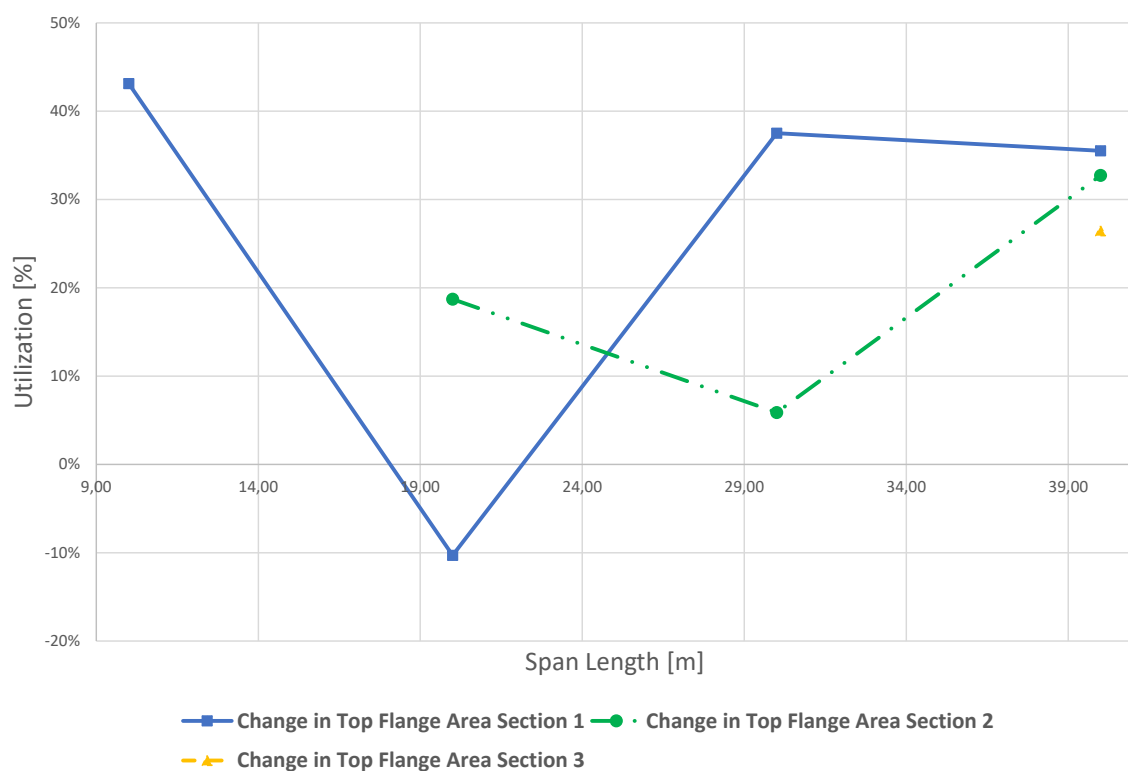
Avarage bot flange thickness [mm]	bfu _1 [mm]	bfu _2 [mm]	bfu _3 [mm]	Avarage bot flange width [mm]	
60	350	350	350	350	
50	250	480	480	403,3333333	
47	510	720	720	650	
60	430	670	770	623,3333333	
60	290	290	290	290	
52	340	380	380	366,6666667	
48	450	600	600	550	
42	510	940	1340	930	
60	290	290	290	290	
53	260	450	450	386,6666667	
55	370	500	500	456,6666667	
38	550	770	1310	876,6666667	
60	290	290	290	290	
40	400	510	510	473,3333333	
35	660	1100	1100	953,3333333	
36	830	540	1110	826,6666667	



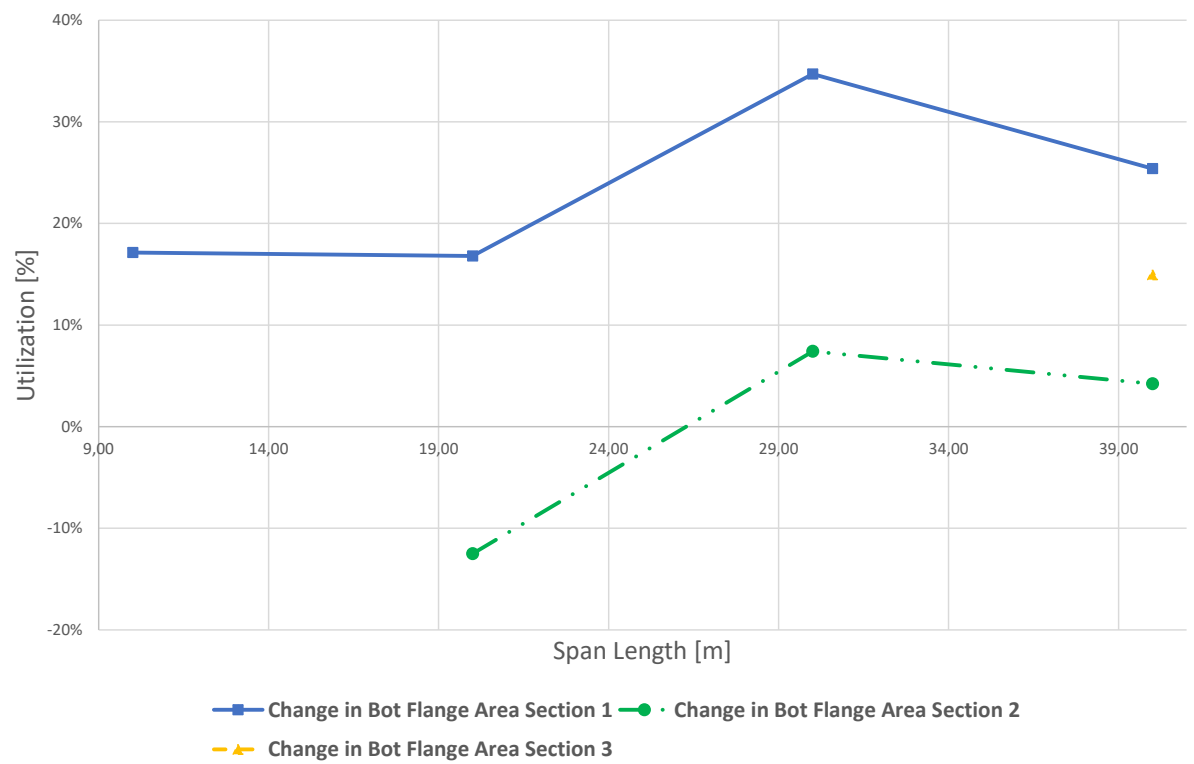




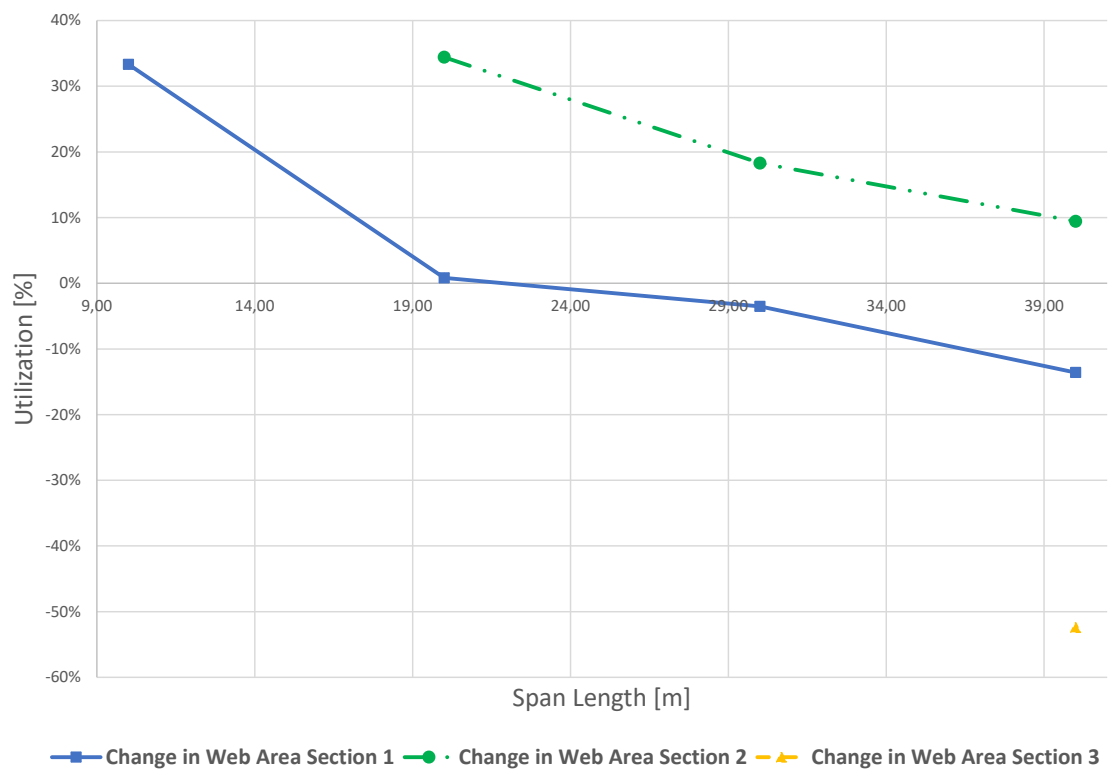
Relative change Top Flange area S460 HFMI treated factory compared to S355 As-welded



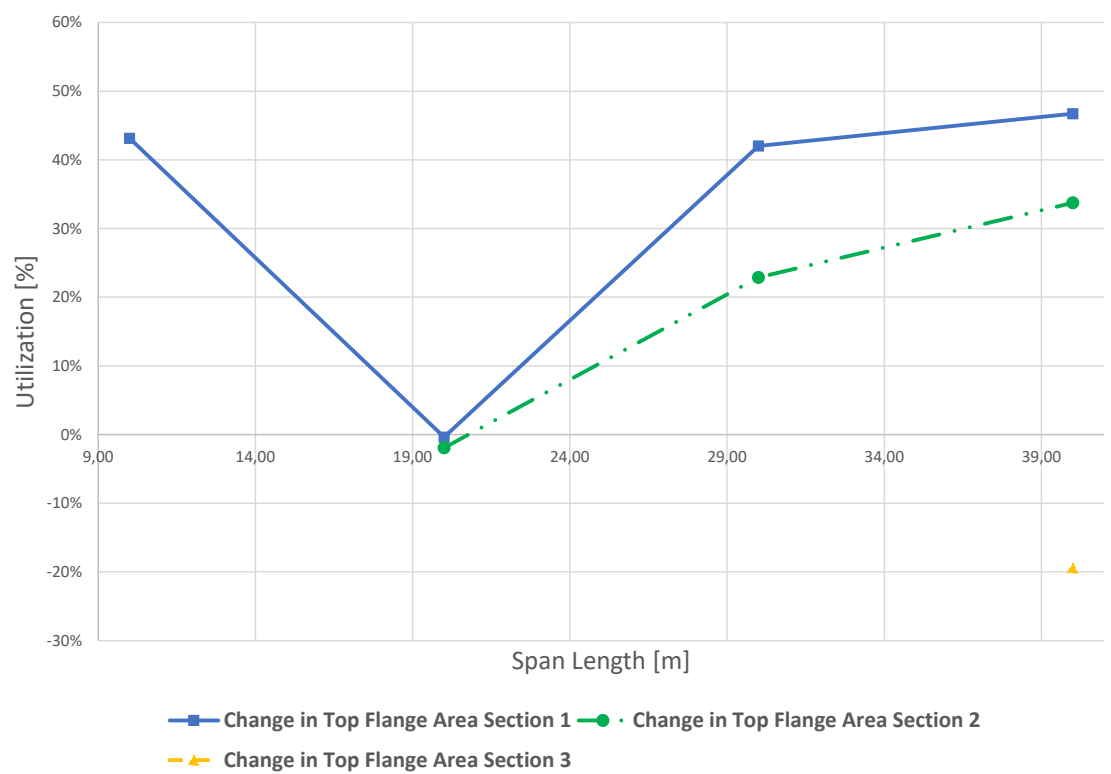
Relative change in Bottom Flange area S460 HFMI treated factory compared to S355 As-welded



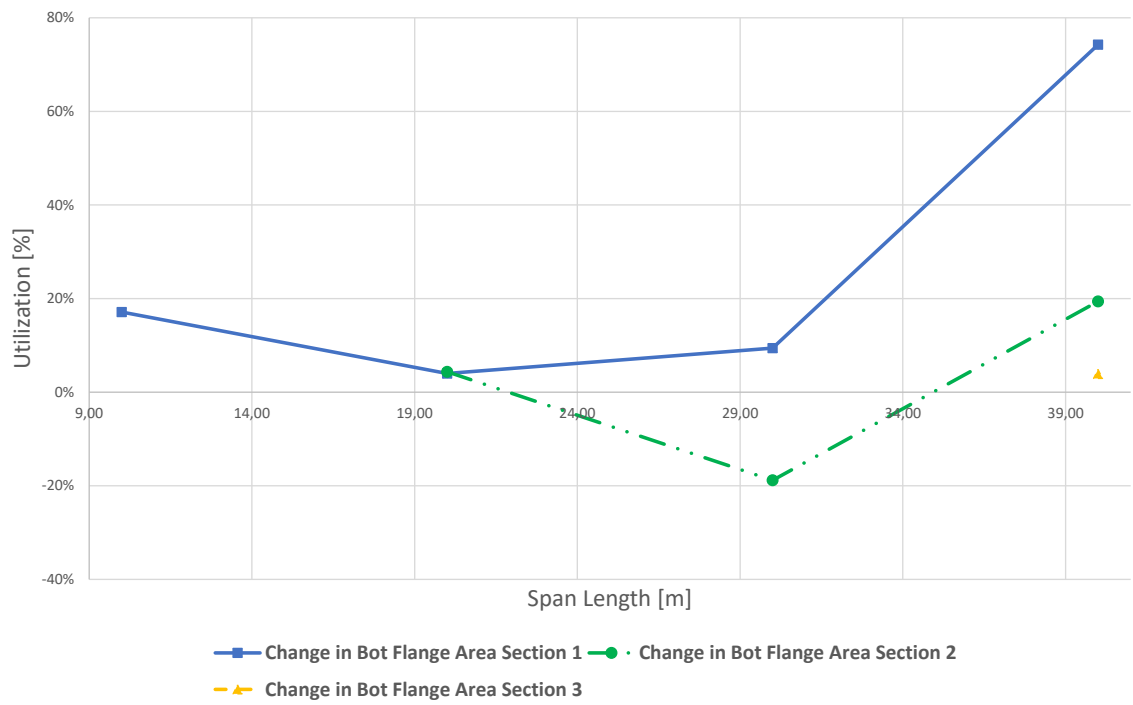
Relative change in Web Plate area S460 HFMI treated factory compared to S355 As-welded

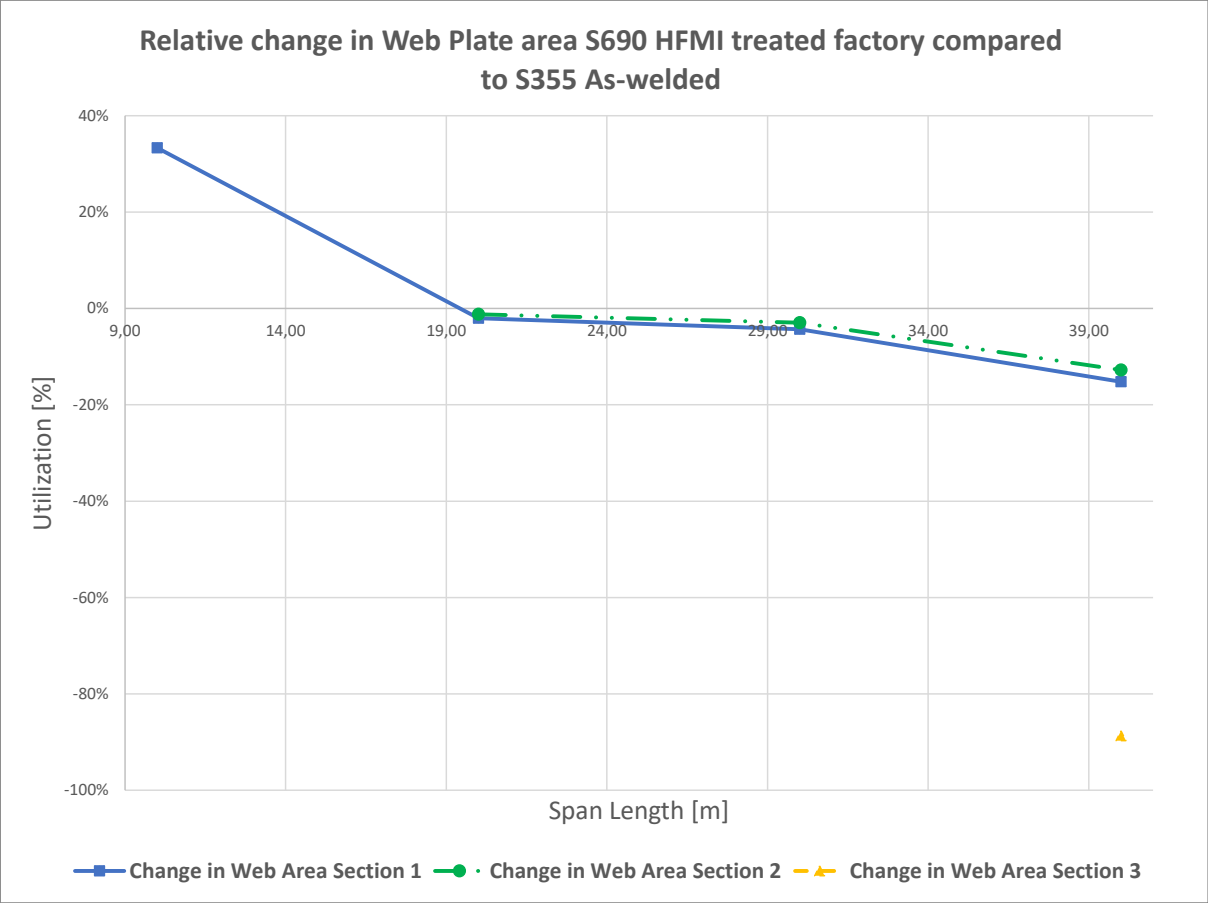


Relative change Top Flange area S690 HFMI treated factory compared to S355 As-welded



Relative change in Bottom Flange area S690 HFMI treated factory compared to S355 As-welded





Comprehensive Project Performance Dashboard - Q3 2024													
Project ID	Project Name	Phase 1: Planning & Design				Phase 2: Development & Testing				Phase 3: Deployment & Monitoring			
		Start Date	End Date	Duration (Days)	Progress (%)	Start Date	End Date	Duration (Days)	Progress (%)	Start Date	End Date	Duration (Days)	Progress (%)
P001	Alpha Initiative	2024-07-01	2024-08-15	45	95%	2024-08-16	2024-09-30	45	80%	2024-10-01	2024-10-31	31	60%
P002	Beta Project	2024-07-05	2024-09-10	70	70%	2024-09-11	2024-11-05	55	50%	2024-11-06	2024-12-15	40	30%
P003	Gamma Task	2024-07-10	2024-08-20	42	100%	2024-08-21	2024-10-10	50	90%	2024-10-11	2024-11-20	40	75%
P004	Delta Mission	2024-07-15	2024-09-25	73	60%	2024-09-26	2024-11-15	50	40%	2024-11-16	2024-12-31	46	20%
P005	Epsilon Project	2024-07-20	2024-09-05	47	85%	2024-09-06	2024-10-25	49	70%	2024-10-26	2024-11-30	35	55%
P006	Zeta Task	2024-07-25	2024-09-15	52	55%	2024-09-16	2024-11-01	46	35%	2024-11-02	2024-12-10	39	15%
P007	Eta Initiative	2024-07-30	2024-09-20	52	90%	2024-09-21	2024-10-31	41	85%	2024-10-31	2024-11-25	25	70%
P008	Theta Project	2024-08-01	2024-10-10	70	40%	2024-10-11	2024-11-20	40	25%	2024-11-21	2024-12-20	30	10%
P009	Iota Task	2024-08-05	2024-09-25	51	75%	2024-09-26	2024-10-20	25	60%	2024-10-21	2024-11-15	25	45%
P010	Kappa Project	2024-08-10	2024-10-05	56	65%	2024-10-06	2024-11-10	35	50%	2024-11-11	2024-12-05	25	35%
P011	Lambda Initiative	2024-08-15	2024-10-20	65	50%	2024-10-21	2024-11-25	35	40%	2024-11-26	2024-12-25	30	25%
P012	Mu Project	2024-08-20	2024-10-15	56	45%	2024-10-16	2024-11-20	35	30%	2024-11-21	2024-12-15	25	20%
P013	Nu Task	2024-08-25	2024-10-25	61	35%	2024-10-26	2024-11-30	35	25%	2024-11-31	2024-12-20	20	15%
P014	Xi Project	2024-09-01	2024-10-31	31	25%	2024-10-31	2024-11-25	25	15%	2024-11-26	2024-12-10	15	10%
P015	Omicron Initiative	2024-09-05	2024-11-05	61	15%	2024-11-06	2024-11-30	25	10%	2024-11-31	2024-12-05	15	5%
P016	Pi Project	2024-09-10	2024-11-10	61	10%	2024-11-11	2024-11-25	15	5%	2024-11-26	2024-11-30	5	0%
P017	Rho Task	2024-09-15	2024-11-15	61	5%	2024-11-16	2024-11-20	5	0%	2024-11-21	2024-11-25	5	0%
P018	Sigma Project	2024-09-20	2024-11-20	61	0%	2024-11-21	2024-11-25	5	0%	2024-11-26	2024-11-30	5	0%
P019	Tau Initiative	2024-09-25	2024-11-25	61	0%	2024-11-26	2024-11-30	5	0%	2024-11-31	2024-11-31	1	0%
P020	Upsilon Project	2024-10-01	2024-11-30	60	0%	2024-11-31	2024-11-31	1	0%	2024-11-31	2024-11-31	1	0%
P021	Phi Task	2024-10-05	2024-11-25	51	0%	2024-11-26	2024-11-30	5	0%	2024-11-31	2024-11-31	1	0%
P022	Chi Project	2024-10-10	2024-11-20	41	0%	2024-11-21	2024-11-25	5	0%	2024-11-26	2024-11-30	5	0%
P023	Psi Initiative	2024-10-15	2024-11-15	31	0%	2024-11-16	2024-11-20	5	0%	2024-11-21	2024-11-25	5	0%
P024	Omega Project	2024-10-20	2024-11-10	21	0%	2024-11-11	2024-11-15	5	0%	2024-11-16	2024-11-20	5	0%
P025	Alpha Task	2024-10-25	2024-11-05	11	0%	2024-11-06	2024-11-10	5	0%	2024-11-11	2024-11-15	5	0%
P026	Beta Project	2024-10-30	2024-11-01	3	0%	2024-11-02	2024-11-05	4	0%	2024-11-06	2024-11-10	5	0%
P027	Gamma Initiative	2024-11-01	2024-11-02	2	0%	2024-11-03	2024-11-04	2	0%	2024-11-05	2024-11-06	2	0%
P028	Delta Task	2024-11-02	2024-11-03	2	0%	2024-11-04	2024-11-05	2	0%	2024-11-06	2024-11-07	2	0%
P029	Epsilon Project	2024-11-03	2024-11-04	2	0%	2024-11-05	2024-11-06	2	0%	2024-11-07	2024-11-08	2	0%
P030	Zeta Initiative	2024-11-04	2024-11-05	2	0%	2024-11-06	2024-11-07	2	0%	2024-11-08	2024-11-09	2	0%
P031	Eta Task	2024-11-05	2024-11-06	2	0%	2024-11-07	2024-11-08	2	0%	2024-11-09	2024-11-10	2	0%
P032	Theta Project	2024-11-06	2024-11-07	2	0%	2024-11-08	2024-11-09	2	0%	2024-11-10	2024-11-11	2	0%
P033	Iota Initiative	2024-11-07	2024-11-08	2	0%	2024-11-09	2024-11-10	2	0%	2024-11-11	2024-11-12	2	0%
P034	Kappa Task	2024-11-08	2024-11-09	2	0%	2024-11-10	2024-11-11	2	0%	2024-11-12	2024-11-13	2	0%
P035	Lambda Project	2024-11-09	2024-11-10	2	0%	2024-11-11	2024-11-12	2	0%	2024-11-13	2024-11-14	2	0%
P036	Mu Initiative	2024-11-10	2024-11-11	2	0%	2024-11-12	2024-11-13	2	0%	2024-11-14	2024-11-15	2	0%
P037	Nu Task	2024-11-11	2024-11-12	2	0%	2024-11-13	2024-11-14	2	0%	2024-11-15	2024-11-16	2	0%
P038	Xi Project	2024-11-12	2024-11-13	2	0%	2024-11-14	2024-11-15	2	0%	2024-11-16	2024-11-17	2	0%
P039	Omicron Initiative	2024-11-13	2024-11-14	2	0%	2024-11-15	2024-11-16	2	0%	2024-11-17	2024-11-18	2	0%
P040	Pi Task	2024-11-14	2024-11-15	2	0%	2024-11-16	2024-11-17	2	0%	2024-11-18	2024-11-19	2	0%
P041	Rho Project	2024-11-15	2024-11-16	2	0%	2024-11-17	2024-11-18	2	0%	2024-11-19	2024-11-20	2	0%
P042	Sigma Initiative	2024-11-16	2024-11-17	2	0%	2024-11-18	2024-11-19	2	0%	2024-11-20	2024-11-21	2	0%
P043	Tau Task	2024-11-17	2024-11-18	2	0%	2024-11-19	2024-11-20	2	0%	2024-11-21	2024-11-22	2	0%
P044	Upsilon Project	2024-11-18	2024-11-19	2	0%	2024-11-20	2024-11-21	2	0%	2024-11-22	2024-11-23	2	0%
P045	Phi Initiative	2024-11-19	2024-11-20	2	0%	2024-11-21	2024-11-22	2	0%	2024-11-23	2024-11-24	2	0%
P046	Chi Task	2024-11-20	2024-11-21	2	0%	2024-11-22	2024-11-23	2	0%	2024-11-24	2024-11-25	2	0%
P047	Psi Project	2024-11-21	2024-11-22	2	0%	2024-11-23	2024-11-24	2	0%	2024-11-25	2024-11-26	2	0%
P048	Omega Initiative	2024-11-22	2024-11-23	2	0%	2024-11-24	2024-11-25	2	0%	2024-11-26	2024-11-27	2	0%
P049	Alpha Task	2024-11-23	2024-11-24	2	0%	2024-11-25	2024-11-26	2	0%	2024-11-27	2024-11-28	2	0%
P050	Beta Project	2024-11-24	2024-11-25	2	0%	2024-11-26	2024-11-27	2	0%	2024-11-28	2024-11-29	2	0%
P051	Gamma Initiative	2024-11-25	2024-11-26	2	0%	2024-11-27	2024-11-28	2	0%	2024-11-29	2024-11-30	2	0%
P052	Delta Task	2024-11-26	2024-11-27	2	0%	2024-11-28	2024-11-29	2	0%	2024-11-30	2024-12-01	2	0%
P053	Epsilon Project	2024-11-27	2024-11-28	2	0%	2024-11-29	2024-11-30	2	0%	2024-12-01	2024-12-02	2	0%
P054	Zeta Initiative	2024-11-28	2024-11-29	2	0%	2024-11-30	2024-12-01	2	0%	2024-12-02	2024-12-03	2	0%
P055	Eta Task	2024-11-29	2024-11-30	2	0%	2024-12-01	2024-12-02	2	0%	2024-12-03	2024-12-04	2	0%
P056	Theta Project	2024-11-30	2024-12-01	2	0%	2024-12-02	2024-12-03	2	0%	2024-12-04	2024-12-05	2	0%
P057	Iota Initiative	2024-12-01	2024-12-02	2	0%	2024-12-03	2024-12-04	2	0%	2024-12-05	2024-12-06	2	0%
P058	Kappa Task	2024-12-02	2024-12-03	2	0%	2024-12-04	2024-12-05	2	0%	2024-12-06	2024-12-07	2	0%
P059	Lambda Project	2024-12-03	2024-12-04	2	0%	2024-12-05	2024-12-06	2	0%	2024-12-07	2024-12-08	2	0%
P060	Mu Initiative	2024-12-04	2024-12-05	2	0%	2024-12-06	2024-12-07	2	0%	2024-12-08	2024-12-09	2	0%
P061	Nu Task	2024-12-05	2024-12-06	2	0%	2024-12-07	2024-12-08	2	0%	2024-12-09	2024-12-10	2	0%
P062	Xi Project	2024-12-06	2024-12-07	2	0%	2024-12-08	2024-12-09	2	0%	2024-12-10	2024-12-11	2	0%
P063	Omicron Initiative	2024-12-07	2024-12-08	2	0%	2024-12-09	2024-12-10	2	0%	2024-12-11	2024-12-12	2	0%
P064	Pi Task	2024-12-08	2024-12-09	2	0%	2024-12-10	2024-12-11	2	0%	2024-12-12	2024-12-13	2	0%
P065	Rho Project	2024-12-09	2024-12-10	2	0%	2024-12-11	2024-12-12	2	0%	2024-12-13	2024-12-14	2	0%
P066	Sigma Initiative	2024-12-10	2024-12-11	2	0%	2024-12-12	2024-12-13	2	0%	2024-12-14	2024-12-15	2	0%
P067	Tau Task	2024-12-11	2024-12-12	2	0%	2024-12-13	2024-12-14	2	0%	2024-12-15	2024-12-16	2	0%
P068	Upsilon Project	2024-12-12	2024-12-13	2	0%	2024-12-14	2024-12-15	2	0%	2024-12-16	2024-12-17	2	0%
P069	Phi Initiative	2024-12-13	2024-12-14	2	0%	2024-12-15	2024-12-16	2	0%	2024-12-17	2024-12-18	2	0%
P070	Chi Task	2024-12-14	2024-12-15	2	0%	2024-12-16	2024-12-17	2	0%	2024-12-18	2024-12-19	2	0%
P071	Psi Project	2024-12-15	2024-12-16	2	0%	2024-12-17	2024-12-18	2	0%	2024-12-19	2024-12-20	2	0%
P072	Omega Initiative	2024-12-16	2024-12-17	2	0%	2024-12-18	2024-12-19	2	0%	2024-12-20	2024-12-21	2	0%
P073	Alpha Task	2024-12-17	2024-12-18	2	0%	2024-12-19	2024-12-20	2	0%	2024-12-21	2024-12-22	2	0%
P074	Beta Project	2024-12-18	2024-12-19	2	0%	2024-12-20	2024-12-21	2	0%	2024-12-22	2024-12-23	2	0%
P075	Gamma Initiative	2024-12-19	2024-12-20	2	0%	2024-12-21	2024-12-22	2	0%	2024-12-23	2024-12-24	2	0%
P076	Delta Task	2024-12-20	2024-12-21	2	0%	2024-12-22	2024-12-23	2	0%	2024-12-24	2024-12-25	2	0%
P077	Epsilon Project	2024-12-21	2024-12-22	2	0%	2024-12-23	2024-12-24	2	0%	2024-12-25	2024-12-26	2	0%
P078	Zeta Initiative	2024-12-22	2024-12-23	2	0%	2024-12-24	2024-12-25	2	0%	2024-12-26	2024-12-27	2	0%
P079	Eta Task	2024-12-23	2024-12-24	2	0%	2024-12-25	2024-12-26	2	0%	2024-12-27	2024-12-28	2	0%
P080	Theta Project	2024-12-24	2024-12-25	2	0%	2024-12-26	2024-12-27	2	0%	2024-12-28	2024-12-29	2	0%
P081	Iota Initiative	2024-12-25	2024-12-26	2	0%	2024-12-27	2024-12-28	2	0%	2024-12-29	2024-12-30	2	0%
P082	Kappa Task	2024-12-26	2024-12-27	2	0%	2024-12-28	2024-12-29	2	0%	2024-12-30	2024-12-31	2	0%
P083	Lambda Project	2024-12-27	2024-12-28	2	0%	2024-12-29	2024-12-30	2	0%	2024-12-31	2025-01-01	2	0%
P084	Mu Initiative	2024-12-28	2024-12-29										

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function		13500	9100	20220	4300	30000	50100	7400	100000	135000	180000	225000	270000	315000	360000	405000	450000	495000	540000	585000	630000	675000	720000	765000	810000	855000	900000	945000	990000	1035000	1080000	1125000	1170000	1215000	1260000	1305000	1350000	1395000	1440000	1485000	1530000	1575000	1620000	1665000	1710000	1755000	1800000	1845000	1890000	1935000	1980000	2025000	2070000	2115000	2160000	2205000	2250000	2295000	2340000	2385000	2430000	2475000	2520000	2565000	2610000	2655000	2700000	2745000	2790000	2835000	2880000	2925000	2970000	3015000	3060000	3105000	3150000	3195000	3240000	3285000	3330000	3375000	3420000	3465000	3510000	3555000	3600000	3645000	3690000	3735000	3780000	3825000	3870000	3915000	3960000	4005000	4050000	4095000	4140000	4185000	4230000	4275000	4320000	4365000	4410000	4455000	4500000	4545000	4590000	4635000	4680000	4725000	4770000	4815000	4860000	4905000	4950000	4995000	5040000	5085000	5130000	5175000	5220000	5265000	5310000	5355000	5400000	5445000	5490000	5535000	5580000	5625000	5670000	5715000	5760000	5805000	5850000	5895000	5940000	5985000	6030000	6075000	6120000	6165000	6210000	6255000	6300000	6345000	6390000	6435000	6480000	6525000	6570000	6615000	6660000	6705000	6750000	6795000	6840000	6885000	6930000	6975000	7020000	7065000	7110000	7155000	7200000	7245000	7290000	7335000	7380000	7425000	7470000	7515000	7560000	7605000	7650000	7695000	7740000	7785000	7830000	7875000	7920000	7965000	8010000	8055000	8100000	8145000	8190000	8235000	8280000	8325000	8370000	8415000	8460000	8505000	8550000	8595000	8640000	8685000	8730000	8775000	8820000	8865000	8910000	8955000	9000000	9045000	9090000	9135000	9180000	9225000	9270000	9315000	9360000	9405000	9450000	9495000	9540000	9585000	9630000	9675000	9720000	9765000	9810000	9855000	9900000	9945000	9990000	10035000	10080000	10125000	10170000	10215000	10260000	10305000	10350000	10395000	10440000	10485000	10530000	10575000	10620000	10665000	10710000	10755000	10800000	10845000	10890000	10935000	10980000	11025000	11070000	11115000	11160000	11205000	11250000	11295000	11340000	11385000	11430000	11475000	11520000	11565000	11610000	11655000	11700000	11745000	11790000	11835000	11880000	11925000	11970000	12015000	12060000	12105000	12150000	12195000	12240000	12285000	12330000	12375000	12420000	12465000	12510000	12555000	12600000	12645000	12690000	12735000	12780000	12825000	12870000	12915000	12960000	13005000	13050000	13095000	13140000	13185000	13230000	13275000	13320000	13365000	13410000	13455000	13500000	13545000	13590000	13635000	13680000	13725000	13770000	13815000	13860000	13905000	13950000	13995000	14040000	14085000	14130000	14175000	14220000	14265000	14310000	14355000	14400000	14445000	14490000	14535000	14580000	14625000	14670000	14715000	14760000	14805000	14850000	14895000	14940000	14985000	15030000	15075000	15120000	15165000	15210000	15255000	15300000	15345000	15390000	15435000	15480000	15525000	15570000	15615000	15660000	15705000	15750000	15795000	15840000	15885000	15930000	15975000	16020000	16065000	16110000	16155000	16200000	16245000	16290000	16335000	16380000	16425000	16470000	16515000	16560000	16605000	16650000	16695000	16740000	16785000	16830000	16875000	16920000	16965000	17010000	17055000	17100000	17145000	17190000	17235000	17280000	17325000	17370000	17415000	17460000	17505000	17550000	17595000	17640000	17685000	17730000	17775000	17820000	17865000	17910000	17955000	18000000	18045000	18090000	18135000	18180000	18225000	18270000	18315000	18360000	18405000	18450000	18495000	18540000	18585000	18630000	18675000	18720000	18765000	18810000	18855000	18900000	18945000	18990000	19035000	19080000	19125000	19170000	19215000	19260000	19305000	19350000	19395000	19440000	19485000	19530000	19575000	19620000	19665000	19710000	19755000	19800000	19845000	19890000	19935000	19980000	20025000	20070000	20115000	20160000	20205000	20250000	20295000	20340000	20385000	20430000	20475000	20520000	20565000	20610000	20655000	20700000	20745000	20790000	20835000	20880000	20925000	20970000	21015000	21060000	21105000	21150000	21195000	21240000	21285000	21330000	21375000	21420000	21465000	21510000	21555000	21600000	21645000	21690000	21735000	21780000	21825000	21870000	21915000	21960000	22005000	22050000	22095000	22140000	22185000	22230000	22275000	22320000	22365000	22410000	22455000	22500000	22545000	22590000	22635000	22680000	22725000	22770000	22815000	22860000	22905000	22950000	22995000	23040000	23085000	23130000	23175000	23220000	23265000	23310000	23355000	23400000	23445000	23490000	23535000	23580000	23625000	23670000	23715000	23760000	23805000	23850000	23895000	23940000	23985000	24030000	24075000	24120000	24165000	24210000	24255000	24300000	24345000	24390000	24435000	24480000	24525000	24570000	24615000	24660000	24705000	24750000	24795000	24840000	24885000	24930000	24975000	25020000	25065000	25110000	25155000	25200000	25245000	25290000	25335000	25380000	25425000	25470000	25515000	25560000	25605000	25650000	25695000	25740000	25785000	25830000	25875000	25920000	25965000	26010000	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Table 1: Performance metrics for various models across different datasets and configurations.																
Model	Dataset A				Dataset B				Dataset C				Dataset D			
	Accuracy	Precision	Recall	F1 Score	Accuracy	Precision	Recall	F1 Score	Accuracy	Precision	Recall	F1 Score	Accuracy	Precision	Recall	F1 Score
Model 1	0.85	0.82	0.88	0.84	0.78	0.75	0.80	0.76	0.92	0.90	0.94	0.91	0.88	0.85	0.89	0.87
	0.86	0.83	0.89	0.85	0.79	0.76	0.81	0.77	0.93	0.91	0.95	0.92	0.89	0.86	0.90	0.88
	0.87	0.84	0.90	0.86	0.80	0.77	0.82	0.78	0.94	0.92	0.96	0.93	0.90	0.87	0.91	0.89
	0.88	0.85	0.91	0.87	0.81	0.78	0.83	0.79	0.95	0.93	0.97	0.94	0.91	0.88	0.92	0.90
Model 2	0.83	0.80	0.86	0.82	0.76	0.73	0.78	0.74	0.90	0.88	0.92	0.89	0.86	0.83	0.87	0.85
	0.84	0.81	0.87	0.83	0.77	0.74	0.79	0.75	0.91	0.89	0.93	0.90	0.87	0.84	0.88	0.86
	0.85	0.82	0.88	0.84	0.78	0.75	0.80	0.76	0.92	0.90	0.94	0.91	0.88	0.85	0.89	0.87
	0.86	0.83	0.89	0.85	0.79	0.76	0.81	0.77	0.93	0.91	0.95	0.92	0.89	0.86	0.90	0.88
Model 3	0.81	0.78	0.84	0.80	0.74	0.71	0.76	0.72	0.88	0.86	0.90	0.87	0.84	0.81	0.85	0.83
	0.82	0.79	0.85	0.81	0.75	0.72	0.77	0.73	0.89	0.87	0.91	0.88	0.85	0.82	0.86	0.84
	0.83	0.80	0.86	0.82	0.76	0.73	0.78	0.74	0.90	0.88	0.92	0.89	0.86	0.83	0.87	0.85
	0.84	0.81	0.87	0.83	0.77	0.74	0.79	0.75	0.91	0.89	0.93	0.90	0.87	0.84	0.88	0.86
Model 4	0.80	0.77	0.83	0.79	0.73	0.70	0.75	0.71	0.87	0.85	0.89	0.86	0.83	0.80	0.84	0.82
	0.81	0.78	0.84	0.80	0.74	0.71	0.76	0.72	0.88	0.86	0.90	0.87	0.84	0.81	0.85	0.83
	0.82	0.79	0.85	0.81	0.75	0.72	0.77	0.73	0.89	0.87	0.91	0.88	0.85	0.82	0.86	0.84
	0.83	0.80	0.86	0.82	0.76	0.73	0.78	0.74	0.90	0.88	0.92	0.89	0.86	0.83	0.87	0.85
Model 5	0.79	0.76	0.82	0.78	0.72	0.69	0.74	0.70	0.86	0.84	0.88	0.85	0.82	0.79	0.83	0.81
	0.80	0.77	0.83	0.79	0.73	0.70	0.75	0.71	0.87	0.85	0.89	0.86	0.83	0.80	0.84	0.82
	0.81	0.78	0.84	0.80	0.74	0.71	0.76	0.72	0.88	0.86	0.90	0.87	0.84	0.81	0.85	0.83
	0.82	0.79	0.85	0.81	0.75	0.72	0.77	0.73	0.89	0.87	0.91	0.88	0.85	0.82	0.86	0.84
Model 6	0.77	0.74	0.80	0.76	0.70	0.67	0.72	0.68	0.84	0.82	0.86	0.83	0.80	0.77	0.81	0.79
	0.78	0.75	0.81	0.77	0.71	0.68	0.73	0.69	0.85	0.83	0.87	0.84	0.81	0.78	0.82	0.80
	0.79	0.76	0.82	0.78	0.72	0.69	0.74	0.70	0.86	0.84	0.88	0.85	0.82	0.79	0.83	0.81
	0.80	0.77	0.83	0.79	0.73	0.70	0.75	0.71	0.87	0.85	0.89	0.86	0.83	0.80	0.84	0.82
Model 7	0.75	0.72	0.78	0.74	0.68	0.65	0.70	0.66	0.82	0.80	0.84	0.81	0.78	0.75	0.79	0.77
	0.76	0.73	0.79	0.75	0.69	0.66	0.71	0.67	0.83	0.81	0.85	0.82	0.79	0.76	0.80	0.78
	0.77	0.74	0.80	0.76	0.70	0.67	0.72	0.68	0.84	0.82	0.86	0.83	0.80	0.77	0.81	0.79
	0.78	0.75	0.81	0.77	0.71	0.68	0.73	0.69	0.85	0.83	0.87	0.84	0.81	0.78	0.82	0.80
Table 2: Summary of model performance across all datasets.																
Model	Dataset A	Dataset B	Dataset C	Dataset D	Avg. Accuracy	Avg. Precision	Avg. Recall	Avg. F1 Score								
Model 1	0.85	0.78	0.92	0.88	0.84	0.82	0.86	0.83								
Model 2	0.83	0.76	0.90	0.86	0.82	0.80	0.84	0.81								
Model 3	0.81	0.74	0.88	0.84	0.80	0.78	0.82	0.79								
Model 4	0.80	0.73	0.87	0.83	0.79	0.77	0.81	0.78								
Model 5	0.79	0.72	0.86	0.82	0.78	0.76	0.80	0.77								
Model 6	0.77	0.70	0.84	0.80	0.76	0.74	0.78	0.75								
Model 7	0.75	0.68	0.82	0.78	0.74	0.72	0.76	0.73								

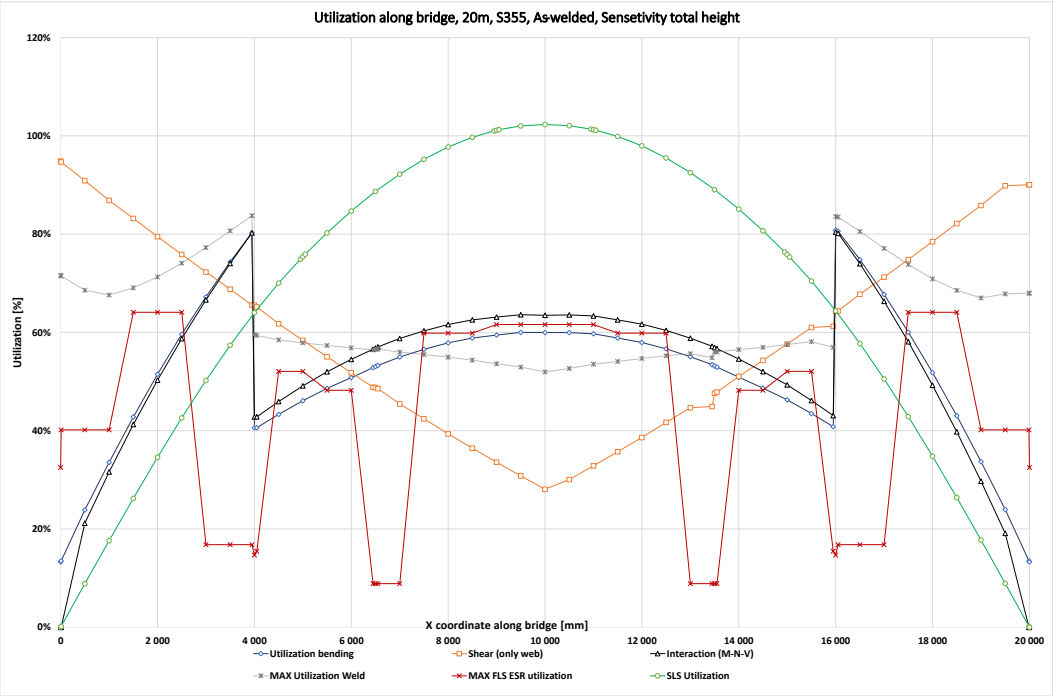
Year of the study	Yes (n cases)	No (n cases)	OR (95% CI)
0	2/90%	4/80%	0.40%
001/01	2/90%	4/80%	0.41%
001/01	2/100%	5/100%	0.41%
1001/01	2/100%	5/100%	0.41%
1001/01	2/90%	4/80%	0.40%
2/001	2/90%	4/80%	0.40%

[illegible]

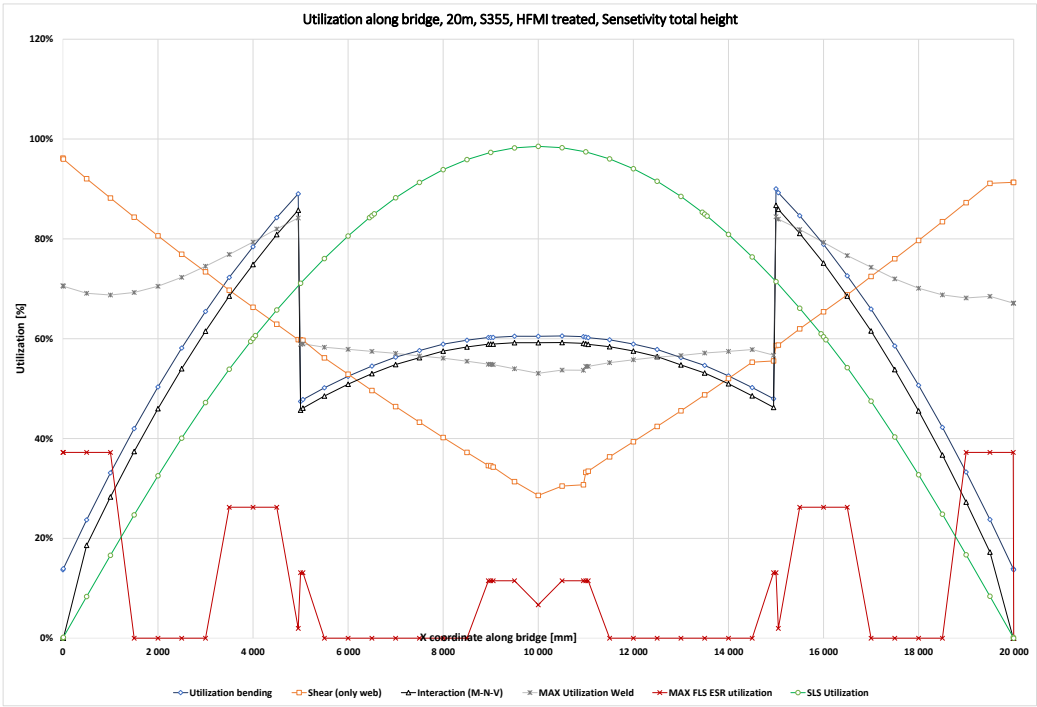
F

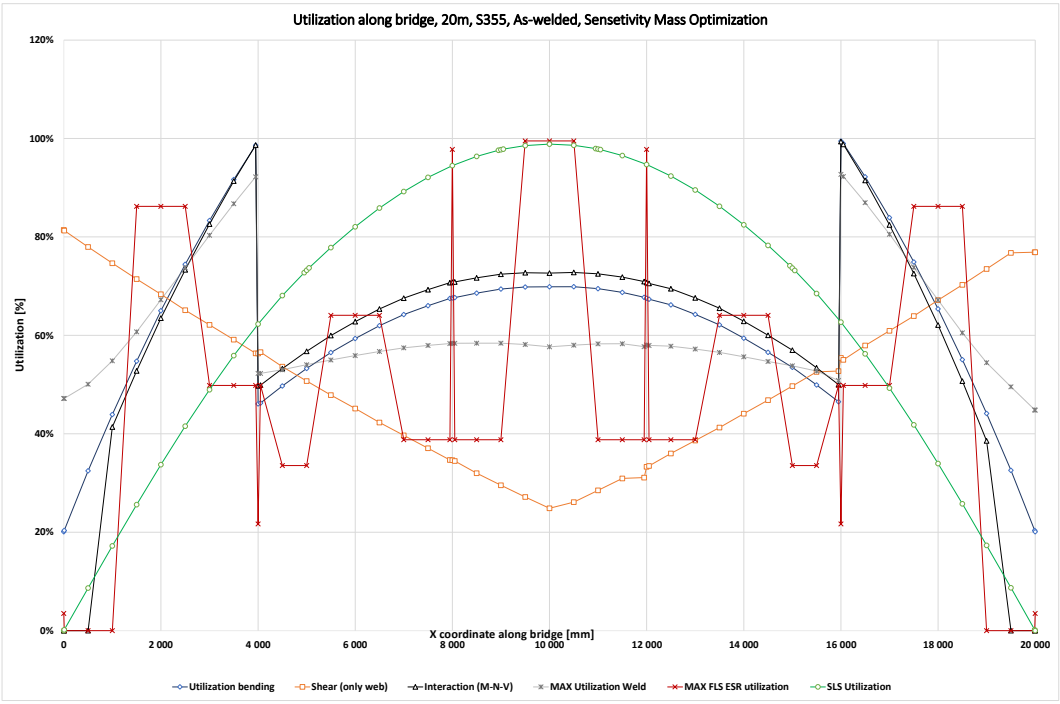
Appendix F

The few sensitivity analyses that have been carried out during the thesis are presented below.



1.000		2.000		3.000		4.000		5.000		6.000		7.000		8.000		9.000		10.000		11.000		12.000		13.000		14.000		15.000		16.000		17.000		18.000		19.000		20.000		21.000		22.000		23.000		24.000		25.000		26.000		27.000		28.000		29.000		30.000		31.000		32.000		33.000		34.000		35.000		36.000		37.000		38.000		39.000		40.000		41.000		42.000		43.000		44.000		45.000		46.000		47.000		48.000		49.000		50.000		51.000		52.000		53.000		54.000		55.000		56.000		57.000		58.000		59.000		60.000		61.000		62.000		63.000		64.000		65.000		66.000		67.000		68.000		69.000		70.000		71.000		72.000		73.000		74.000		75.000		76.000		77.000		78.000		79.000		80.000		81.000		82.000		83.000		84.000		85.000		86.000		87.000		88.000		89.000		90.000		91.000		92.000		93.000		94.000		95.000		96.000		97.000		98.000		99.000		100.000	
1.000	2.000	3.000	4.000	5.000	6.000	7.000	8.000	9.000	10.000	11.000	12.000	13.000	14.000	15.000	16.000	17.000	18.000	19.000	20.000	21.000	22.000	23.000	24.000	25.000	26.000	27.000	28.000	29.000	30.000	31.000	32.000	33.000	34.000	35.000	36.000	37.000	38.000	39.000	40.000	41.000	42.000	43.000	44.000	45.000	46.000	47.000	48.000	49.000	50.000	51.000	52.000	53.000	54.000	55.000	56.000	57.000	58.000	59.000	60.000	61.000	62.000	63.000	64.000	65.000	66.000	67.000	68.000	69.000	70.000	71.000	72.000	73.000	74.000	75.000	76.000	77.000	78.000	79.000	80.000	81.000	82.000	83.000	84.000	85.000	86.000	87.000	88.000	89.000	90.000	91.000	92.000	93.000	94.000	95.000	96.000	97.000	98.000	99.000	100.000																																																																																																				



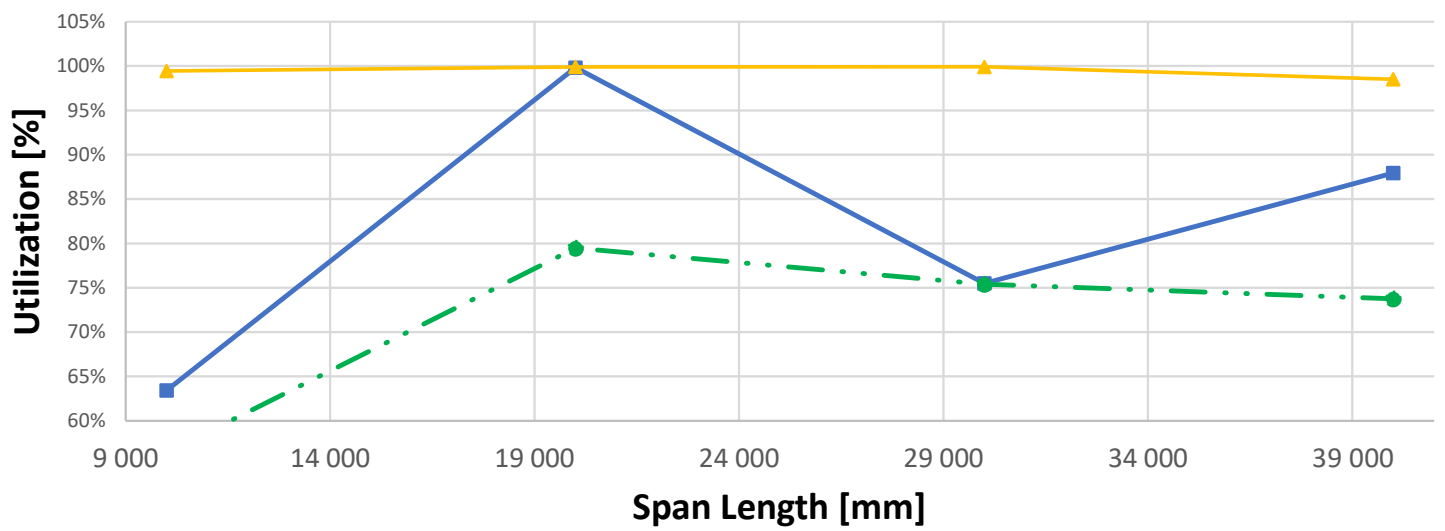


G

Appendix G

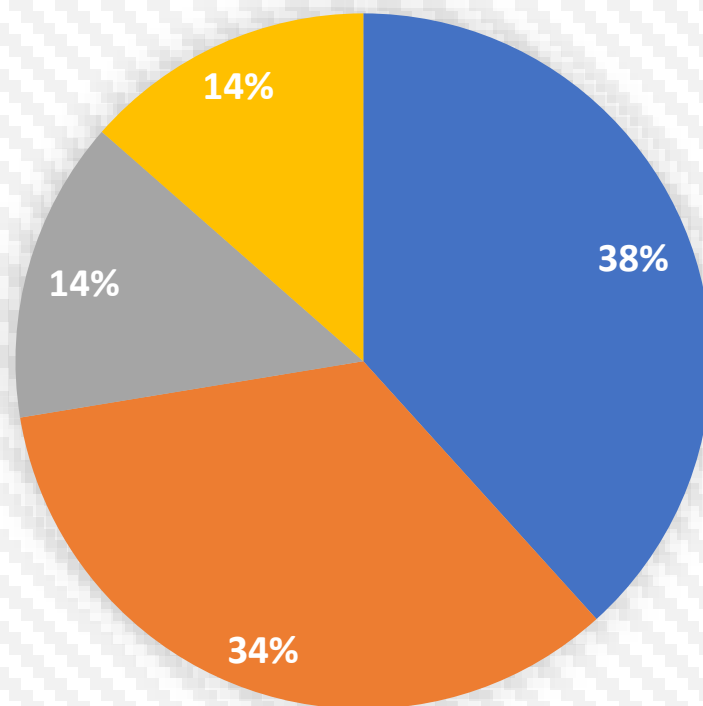
Although the aim of the work was not to confirm potential using the λ -method, some smaller and simpler tests were carried out for this purpose as well. These are presented below.

Utilization ratio S355 As-welded



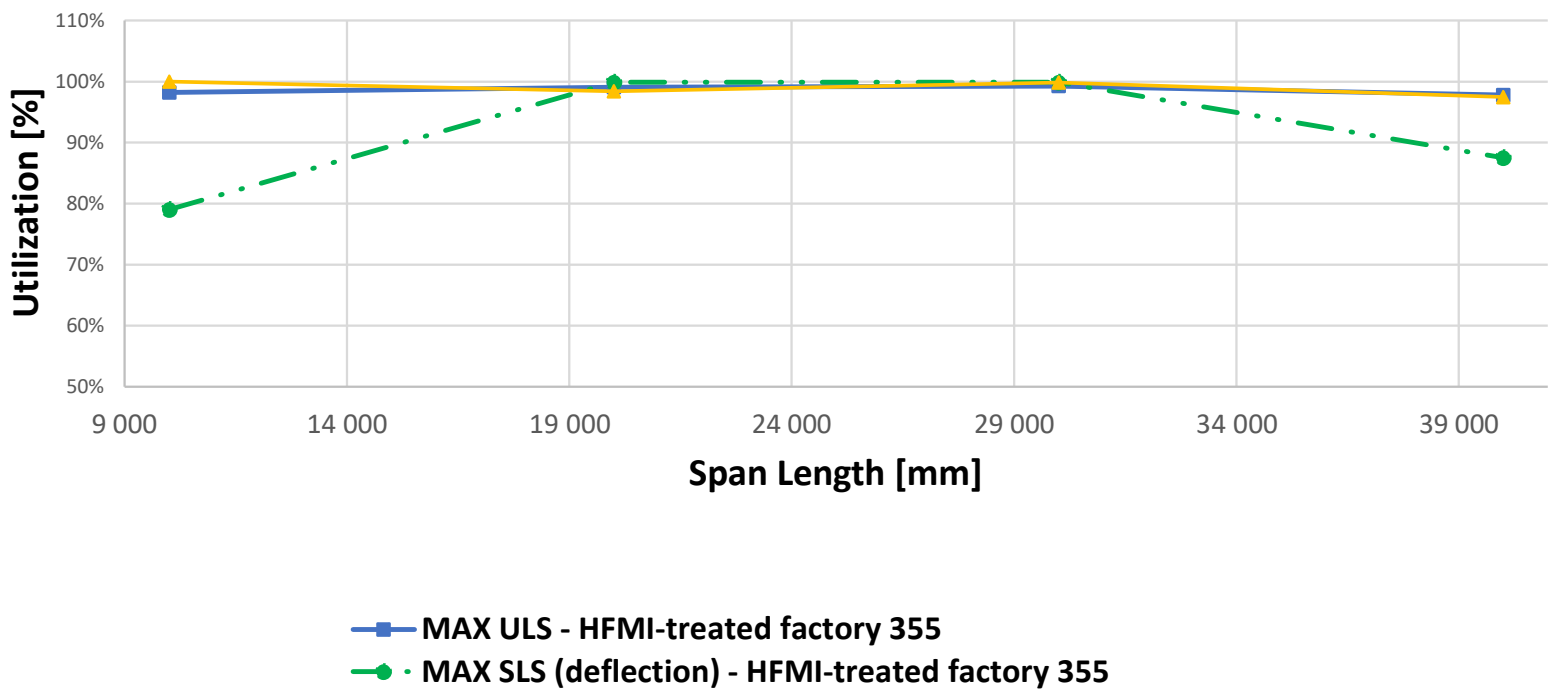
—■— MAX ULS - As-welded 355

As-welded 355 Standard train mix for 40m span

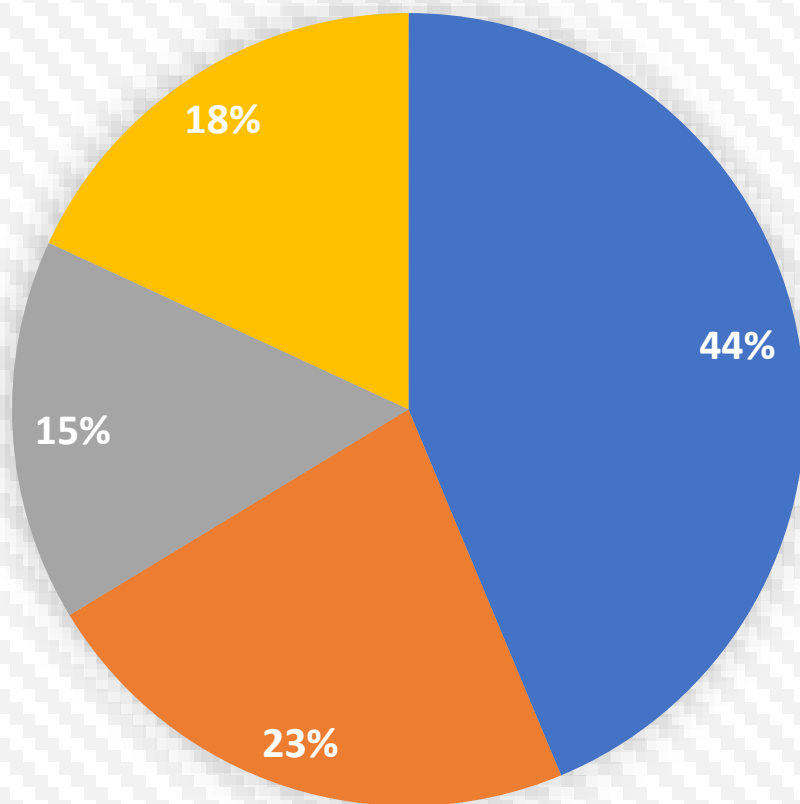


■ Material [SEK] ■ painting [SEK] ■ Welding [SEK] ■ Total Other Cost [SEK]

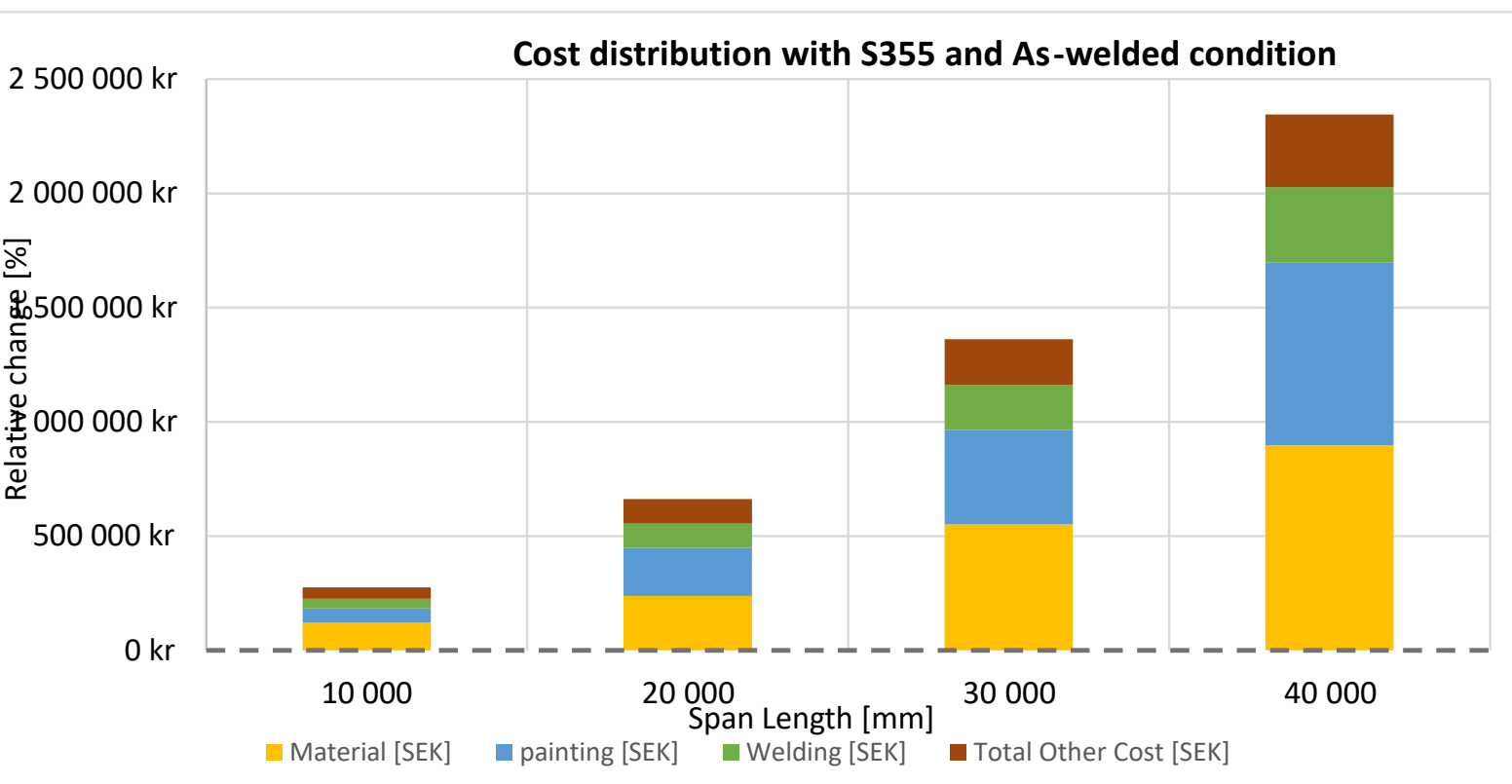
Utilization ratio S355 HFMI treated



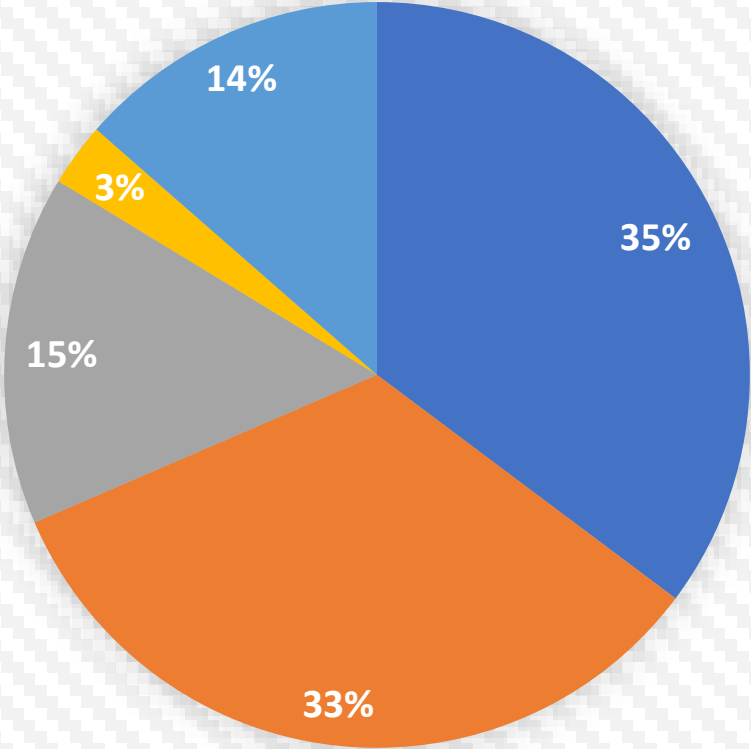
As-welded 355 Standard train mix for 10m span



■ Material [SEK] ■ painting [SEK] ■ Welding [SEK] ■ Total Other Cost [SEK]

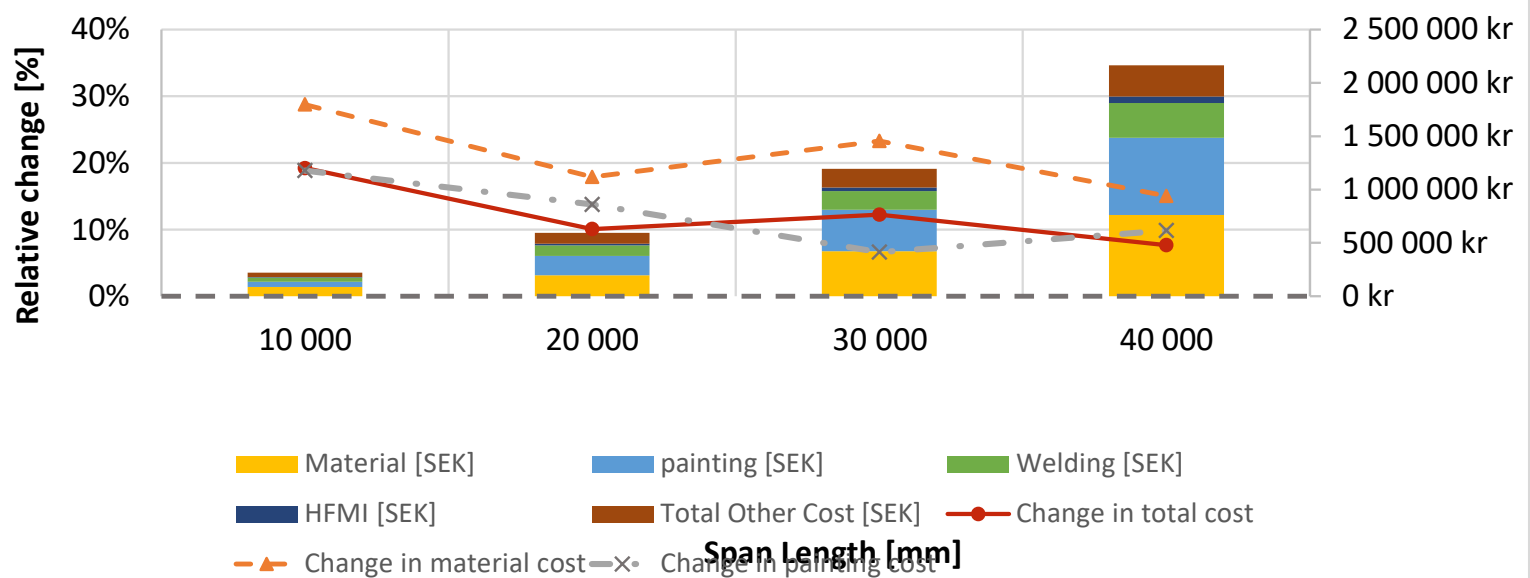


HFMI-treated factory 355 Standard train mix for 40m span

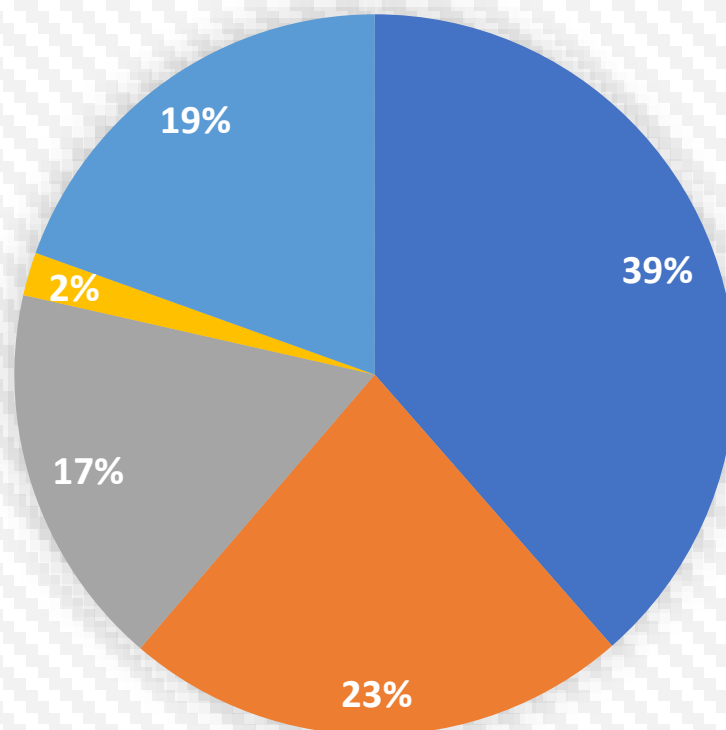


■ Material [SEK] ■ painting [SEK] ■ Welding [SEK] ■ HFMI [SEK] ■ Total Other Cost [SEK]

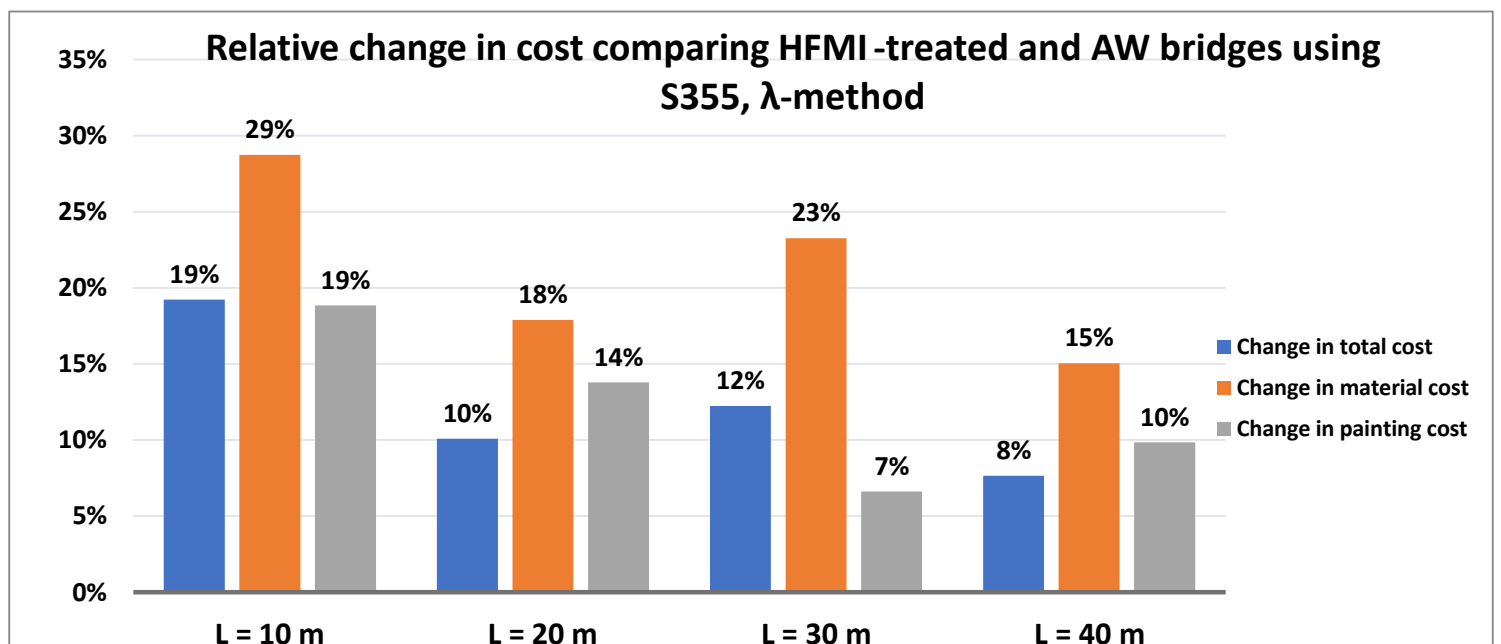
Relative change in cost S355 HFMI treated factory compared to S355 As-welded using simplified LAMBDA METHOD



HFMI-treated factory 355 Standard train mix for 10m span



■ Material [SEK] ■ painting [SEK] ■ Welding [SEK] ■ HFMI [SEK] ■ Total Other Cost [SEK]



Bridge data string	1330	9	2222,22	4	140	909	2	4000	7000	13000	16000	35	40	40	370	600	600	12	12
20 m HFMI treated S355, ULS and SLS																			
X_coord [mm]	Utilization bending	Global buckling factor	"Simplified LT-buckling"	Flange induced buckling	Shear (only web)	Interaction (M-N-V)	Weld bot (long shear)	Weld top (shear)	Weld top (local buckling from patch load)	Weld top (interaction n patch-moment)	Weld top (VON misses yielding)	UPE-beam (worst)	L-profile (worst)	Vertical deflection [mm]	Eigen frequency UR	SLS Utilization	MAX FLS LAMBDA utilization n	MAX ULS	MAX Utilization Weld
0	23,21%	1,35	21,60%	29,14%	83,02%	0,00%	51,83%	46,33%	16,83%	17,69%	48,20%	27,06%	91,80%	0,00	90,18%	0,00%	37,21%	83,02%	51,83%
10	23,45%	1,35	21,60%	29,14%	82,88%	0,00%	51,75%	46,25%	16,83%	17,86%	48,19%			0,06		0,17%	74,20%	82,88%	51,75%
500	34,93%	1,35	21,60%	29,14%	79,47%	0,00%	49,62%	44,34%	16,83%	25,65%	50,49%			2,83		8,48%	74,20%	79,47%	50,49%
1000	45,81%	1,35	21,60%	29,14%	76,19%	41,81%	47,57%	42,52%	16,83%	32,97%	54,57%			5,62		16,87%	74,20%	76,19%	54,57%
1500	56,01%	1,35	21,61%	29,14%	72,74%	52,84%	45,42%	40,59%	16,83%	39,80%	59,58%			8,37		25,10%	63,58%	72,74%	59,58%
2000	65,70%	1,35	21,61%	29,14%	69,58%	63,27%	43,44%	38,82%	16,83%	46,28%	65,33%			11,02		33,07%	63,58%	69,58%	65,33%
2500	74,74%	1,35	47,55%	29,14%	66,40%	72,96%	41,46%	37,05%	16,83%	52,32%	71,25%			13,57		40,72%	63,58%	74,74%	71,25%
3000	83,16%	1,35	47,55%	29,14%	63,28%	82,01%	39,51%	35,31%	16,83%	57,94%	77,16%			15,99		47,97%	63,58%	83,16%	77,16%
3500	91,03%	1,35	47,55%	29,14%	60,23%	90,53%	37,61%	33,61%	16,83%	63,21%	82,96%			18,25		54,76%	79,42%	91,03%	82,96%
3950	97,63%	1,35	47,55%	29,14%	57,37%	97,70%	35,82%	32,01%	16,83%	67,63%	87,94%			20,14		60,42%	79,42%	97,70%	87,94%
4000	57,77%	1,35	16,62%	39,04%	57,34%	57,59%	40,54%	34,81%	15,34%	42,48%	58,97%			20,34		61,02%	79,42%	58,97%	58,97%
4050	58,15%	1,35	16,62%	39,04%	58,05%	57,99%	40,42%	34,71%	15,34%	42,73%	59,19%			20,54		61,62%	75,20%	59,19%	59,19%
4500	61,65%	1,35	23,09%	39,04%	55,03%	61,92%	38,32%	32,90%	15,34%	44,98%	60,80%			22,28		66,83%	70,94%	61,92%	60,80%
5000	65,26%	1,35	23,09%	39,04%	52,05%	66,01%	36,25%	31,12%	15,34%	47,30%	62,68%			24,09		72,26%	70,94%	66,01%	62,68%
5500	68,51%	1,35	23,09%	39,04%	49,14%	69,70%	34,21%	29,38%	15,34%	49,39%	64,45%			25,76		77,29%	70,94%	69,70%	64,45%
6000	71,34%	1,35	23,09%	39,04%	46,28%	72,94%	32,23%	27,67%	15,34%	51,22%	66,02%			27,29		81,87%	85,41%	72,94%	66,02%
6500	73,93%	1,35	23,09%	39,04%	43,50%	75,89%	30,29%	26,01%	15,34%	52,90%	67,50%			28,66		85,98%	85,41%	75,89%	67,50%
6950	76,00%	1,35	26,91%	39,04%	40,98%	78,24%	28,53%	24,50%	15,34%	54,25%	68,69%			29,75		89,26%	89,26%	78,24%	68,69%
7000	76,13%	1,35	26,89%	39,04%	40,94%	78,40%	28,51%	24,48%	15,34%	54,34%	68,79%			29,87		89,60%	89,26%	78,40%	68,79%
7050	76,27%	1,35	26,89%	39,04%	40,78%	78,56%	28,39%	24,38%	15,34%	54,43%	68,87%			29,98		89,93%	89,26%	78,56%	68,87%
7500	77,86%	1,35	26,89%	39,04%	38,03%	80,37%	26,48%	22,74%	15,34%	55,48%	69,68%			30,90		92,69%	89,26%	80,37%	69,68%
8000	79,27%	1,35	26,89%	39,04%	35,46%	81,98%	24,69%	21,20%	15,34%	56,41%	70,41%			31,75		95,25%	96,31%	81,98%	70,41%
8500	80,30%	1,35	26,89%	39,04%	32,90%	83,16%	22,91%	19,67%	15,34%	57,11%	70,86%			32,42		97,25%	96,31%	83,16%	70,86%
9000	81,02%	1,35	27,98%	39,04%	30,40%	83,99%	21,17%	18,18%	15,34%	57,62%	71,09%			32,89		98,68%	96,31%	83,99%	71,09%
9500	81,12%	1,35	27,98%	39,04%	27,97%	84,12%	19,48%	16,72%	15,34%	57,73%	70,81%			33,18		99,53%	97,70%	84,12%	70,81%
10000	81,18%	1,35	27,98%	39,04%	25,62%	84,21%	17,84%	15,32%	15,34%	57,83%	70,55%			33,27		99,80%	97,70%	84,21%	70,55%
10500	81,37%	1,35	27,98%	39,04%	26,81%	84,40%	18,67%	16,03%	15,34%	57,89%	70,87%			33,16		99,49%	97,70%	84,40%	70,87%
11000	80,91%	1,35	27,98%	39,04%	29,25%	83,87%	20,37%	17,49%	15,34%	57,55%	70,82%			32,86		98,59%	96,31%	83,87%	70,82%
11500	80,46%	1,35	26,93%	39,04%	31,74%	83,34%	22,10%	18,98%	15,34%	57,21%	70,81%			32,37		97,12%	96,31%	83,34%	70,81%
12000	79,37%	1,35	26,93%	39,04%	34,30%	82,09%	23,88%	20,51%	15,34%	56,48%	70,29%			31,69		95,08%	96,31%	82,09%	70,29%
12500	77,97%	1,35	26,93%	39,04%	36,86%	80,50%	25,67%	22,04%	15,34%	55,54%	69,55%			30,83		92,50%	89,26%	80,50%	69,55%
12950	76,44%	1,35	26,93%	39,04%	37,03%	78,75%	25,78%	22,14%	15,34%	54,54%	68,26%			29,90		89,71%	89,26%	78,75%	68,26%
13000	76,19%	1,35	26,95%	39,04%	39,48%	78,46%	27,49%	23,61%	15,34%	54,37%	68,54%			29,79		89,38%	89,26%	78,46%	68,54%
13050	75,94%	1,35	26,95%	39,04%	39,65%	78,18%	27,61%	23,71%	15,34%	54,21%	68,36%			29,68		89,04%	89,26%	78,18%	68,36%
13500	74,10%	1,35	23,11%	39,04%	42,33%	76,08%	29,47%	25,31%	15,34%	53,01%	67,37%			28,58		85,75%	85,41%	76,08%	67,37%
14000	71,43%	1,35	23,11%	39,04%	45,12%	73,04%	31,42%	26,98%	15,34%	51,28%	65,80%			27,21		81,63%	85,41%	73,04%	65,80%
14500	68,56%	1,35	23,11%	39,04%	47,98%	69,77%	33,41%	28,69%	15,34%	49,43%	64,18%			25,68		77,05%	70,94%	69,77%	64,18%
15000	65,50%	1,35	23,11%	39,04%	50,89%	66,27%	35,44%	30,43%	15,34%	47,45%	62,53%			24,01		72,03%	70,94%	66,27%	62,53%
15500	61,88%	1,35	23,11%	39,04%	53,85%	62,13%	37,50%	32,20%	15,34%	45,12%	60,60%			22,21		66,62%	70,94%	62,13%	60,60%
15950	58,43%	1,35	16,73%	39,04%	54,02%	58,18%	37,62%	32,30%	15,34%	42,90%	58,02%			20,47		61,42%	75,20%	58,43%	58,02%
16000	98,79%	1,35	47,88%	29,14%	56,76%	98,67%	34,91%	31,20%	16,83%	68,40%	88,65%			20,27		60,82%	79,42%	98,79%	88,65%
16050	97,94%	1,35	47,88%	29,14%	56,05%	97,70%	35,00%	31,28%	16,83%	67,84%	87,95%			20,07		60,22%	79,42%	97,94%	87,95%
16500	91,56%	1,35	47,88%	29,14%	59,04%	90,46%	36,86%	32,94%	16,83%	63,56%	83,13%			18,19		54,57%	79,42%	91,56%	83,13%
17000	83,70%	1,35	47,88%	29,14%	62,10%	81,52%	38,77%	34,65%	16,83%	58,30%	77,30%			15,93		47,79%	63,58%	83,70%	77,30%
17500	75,20%	1,35	47,88%	29,14%	65,22%	71,82%	40,72%	36,39%	16,83%	52,62%	71,28%			13,52		40,56%	63,58%	75,20%	71,28%
18000	66,08%	1,35	21,74%	29,14%	68,40%	61,36%	42,70%	38,17%	16,83%	46,53%	65,22%			10,98		32,93%	63,58%	68,40%	65,22%
18500	56,29%	1,35	21,74%	29,14%	71,56%	50,09%	44,68%	39,93%	16,83%	39,99%	59,33%			8,33		24,99%	63,58%	71,56%	59,33%
19000	45,99%	1,35	21,73%	29,14%	75,01%	38,15%	46,83%	41,86%	16,83%	33,09%	54,16%			5,60		16,80%	74,20%	75,01%	54,16%
19500	35,02%	1,35	21,73%	29,14%	78,28%	0,00%	48,88%	43,68%	16,83%	25,71%	49,96%			2,81		8,44%	74,20%	78,28%	49,96%
19990	23,44%	1,35	21,73%	29,14%	78,43%	0,00%	48,97%	43,76%	16,83%	17,85%	45,80%			0,06		0,17%	74,20%	78,43%	48,97%
20000	23,21%	1,35	9,60%	29,14%	78,43%	0,00%	48,97%	43,76%	16,83%	17,69%	45,74%			0,00		0,00%	37,21%	78,43%	48,97%

12,00]	1235,00]	1215,00]	1215,00]	35,00]	40,00]	40,00]	340,00]	740,00]	740,00]	20000,00]	20000,00]	1150,00]	1150,00]	58,00]	50,00]	0,00]	factory
20 m As-welded S355, FltSimplified Lambda																	
X Flt 1		X Flt 2		Detail 2		X Flt 3		Detail 3		X Flt 4		Detail 4		X Flt 5		Detail 5	
3999,00	0,30	3999,00	0,51	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,56	0,94	10,00	0,56
4001,00	0,30	4001,00	0,50	2222,20	0,88	2222,20	0,81	2222,20	0,94	3999,00	0,94	4001,00	0,82	3999,00	0,82	4001,00	0,64
9499,00	0,47	9499,00	0,78	4444,40	0,70	4444,40	0,69	4444,40	0,82	4001,00	0,82	4001,00	0,93	9499,00	0,93		0,65
9501,00	0,47	9501,00	0,78	6666,70	0,88	6666,70	0,86	6666,70	0,93	9499,00	0,93		0,93				4001,00
10000,00	0,47	10000,00	0,79	8888,90	0,99	8888,90	0,97	8888,90	1,01	9501,00	1,01		1,01				3999,00
#N/A		#N/A		#N/A		#N/A		#N/A		#N/A		#N/A		#N/A		#N/A	
10000,00	0,47	10000,00	0,79	1111,10	0,99	1111,10	0,97	1111,10	1,01	10499,00	1,01	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
10499,00	0,47	10499,00	0,78	1333,30	0,88	1333,30	0,86	1333,30	0,93	15999,00	0,93	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
10501,00	0,47	10501,00	0,78	1555,60	0,70	1555,60	0,69	1555,60	0,82	15999,00	0,82	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
15999,00	0,30	15999,00	0,50	1777,80	0,88	1777,80	0,81	1777,80	0,94	16001,00	0,94	16001,00	0,64	15999,00	0,64	16001,00	0,71
16001,00	0,30	16001,00	0,51	20000,00	0,00	20000,00	0,00	20000,00	0,56	19990,00	0,56	16001,00	0,72	15999,00	0,72	15999,00	0,84

20 m As-welded S355, Eigen frequency [Hz]	
Lower bound	n_0
	Upper bound

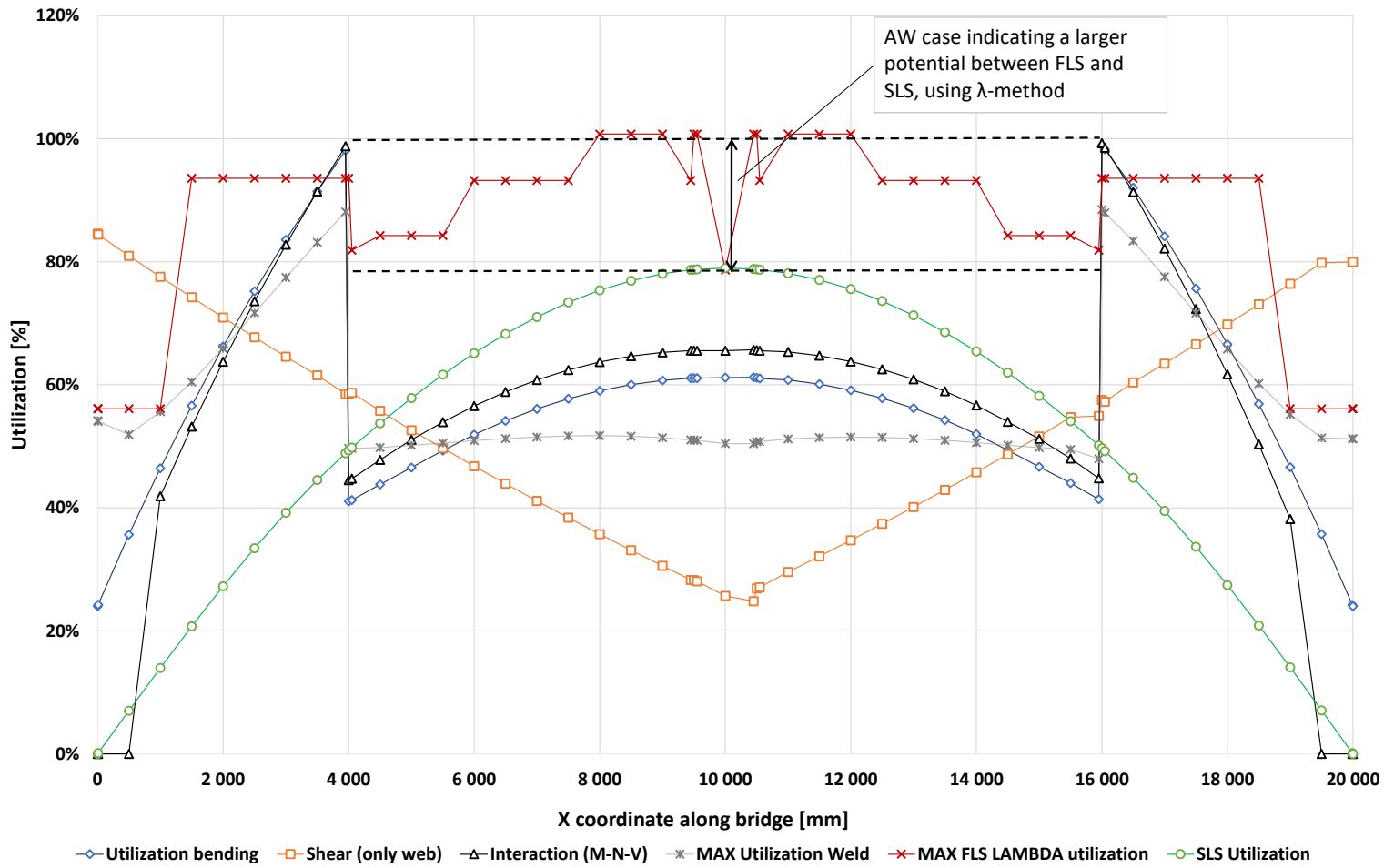
Total [SEK]	Material [SEK]	painting [SEK]	Welding [SEK]	HFMI [SEK]	Total O'other Cost [SEK]	Change in total cost	Change in material cost	Change in painting cost	Change in welding cost	Change in "other costs"	Weight plates [kg]	Weight profiles [kg]	Weight Total [kg]	Change in weight of hot-rolled plates	Weight profiles [%]	Change in total weight	CO ₂ [kg]
Top Flange Area Section 1	Top Flange Area Section 2	Top Flange Area Section 3	Bot Flange Area Section 1	Bot Flange Area Section 2	Bot Flange Area Section 3	Web Area Section 1	Web Area Section 2	Web Area Section 3	Change in Top Flange Area Section 1	Change in Top Flange Area Section 2	Change in Top Flange Area Section 3	Change in Bot Flange Area Section 1	Change in Bot Flange Area Section 2	Change in Bot Flange Area Section 3	Change in Web Area Section 1	Change in Web Area Section 2	Change in Web Area Section 3

X plate plot		Top Flange	Bottom Flange	Web Plate
0	0,00%	0,00%	0,00%	0,00%
2,00	0,00%	0,00%	0,00%	0,00%
2,00	0,00%	0,00%	0,00%	0,00%
10500,00	0,00%	0,00%	0,00%	0,00%
10500,00	0,00%	0,00%	0,00%	0,00%
20000	0,00%	0,00%	0,00%	0,00%

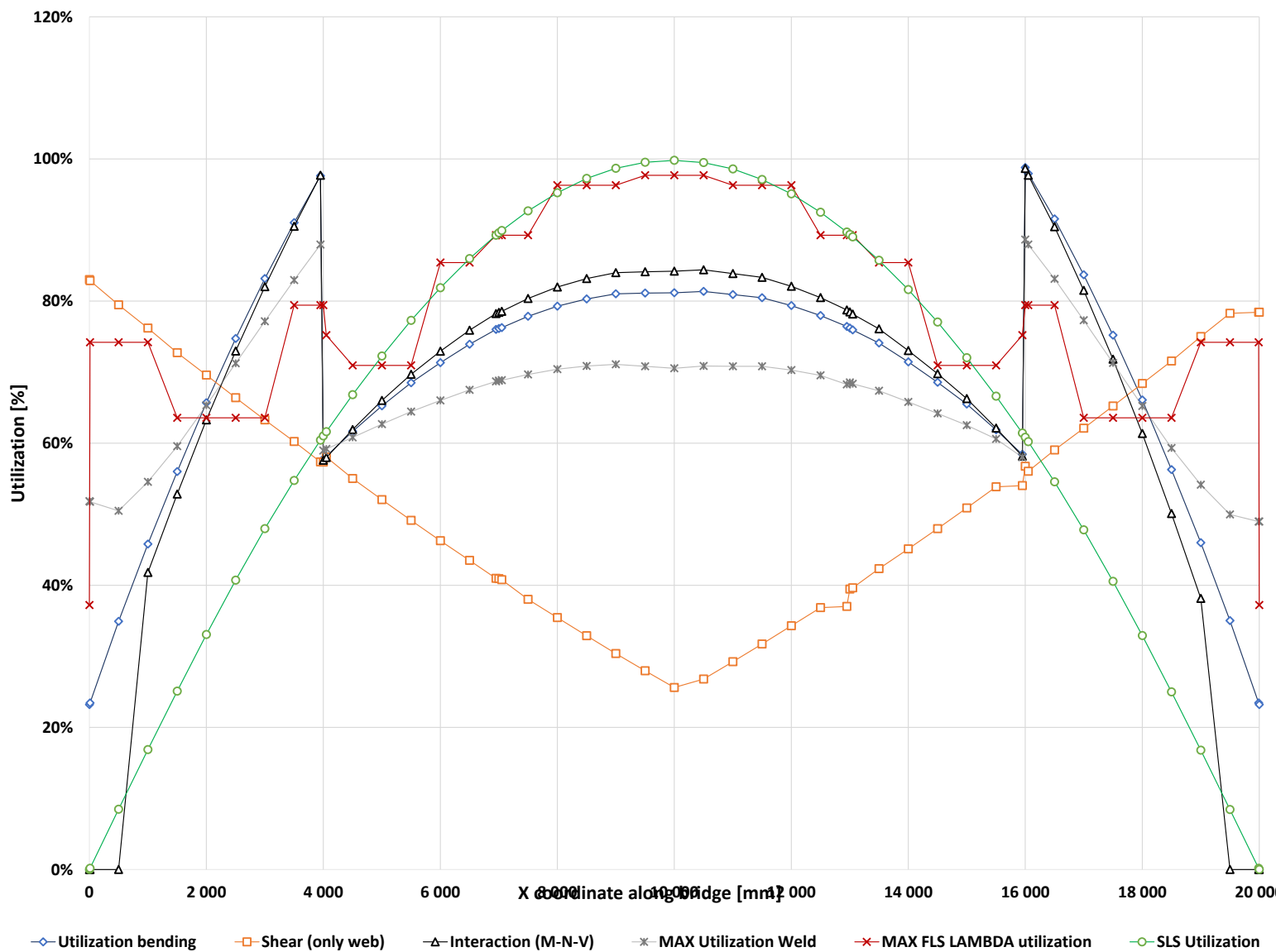
20 m HFMI treated S355, Eigen frequency [Hz]
Lower bound n_0 Upper bound

[illegible]

Utilization along bridge, 20m, S355, As-welded λ -method



Utilization along bridge, 20m, S355, HFMI λ -method



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