THESIS FOR THE DEGREE OF LICENTIATE OF ENGINEERING

Performance of high-frequency mechanical impact treatment for bridge application

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CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover:
Image of a bridge in Sweden and HFMI treatment of a fatigue test specimen photographed and created by the author.

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To Caroline, Miryam and Meysam

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ABSTRACT

This thesis investigates the performance of high-frequency mechanical impact (HFMI) treatment for implementation on new bridge designs. Fatigue strength improvement with HFMI can enable lightweight design of bridges and allow the utilisation of the benefits of high-strength steels. Experimental work on HFMI-treated joints with thick main plates relevant for bridges are, however, scarce and comprehensive studies on the thickness effect few. Studies of various bridge types were performed in this thesis showing that 20% material saving is possible in the load-carrying members through post-weld treatment and the use of high-strength steel ($f_y > 355$ MPa) where necessary. Limitations of the application of HFMI treatment on bridges were also identified regarding the degree of improvement and choice of steel grades.

The thickness effect was studied on the basis of an established database of 582 fatigue test results of different types of HFMI-treated joints from 28 studies. It was shown that the thickness effect becomes weaker than what is recommended for as-welded joints. Fatigue experiments were conducted on a typical fatigue-prone detail in bridges with load-carrying plates of 40 and 60 mm which showed a significant fatigue strength improvement as a result of HFMI treatment, exceeding recommended fatigue strengths given by the International Institute of Welding. Based on the fatigue experiments, a weak thickness effect was derived for non-load-carrying transverse attachment joints where the attachment and weld sizes are kept constant.

Keywords: fatigue; thickness effect; size effect; bridge; steel; HFMI; LEFM

Preface

The work in this thesis was carried out between January 2015 and June 2017 in Gothenburg, Sweden, as a collaboration between the company ELU Konsult AB and the Division of Structural Engineering at Chalmers University of Technology. The funding was provided in part by the Swedish Road Administration together with the Swedish research agency Vinnova and in part by the Norwegian Public Road Administration.

I want to express my gratitude and appreciation to my supervisor, Associate Professor Mohammad Al-Emrani, who already during my Masters' studies inspired me to want to work with research. I thank him for his engagement on a detailed level in the planning, execution and writing processes involved in this thesis and his valuable advice and discussions. I would also like to thank my co-supervisors during this period, Doctor Farshid Zamiri and Assistant Professor Halid Can Yıldırım, for their tireless involvements and encouraging words.

Many thanks to all the partners in the Vinnova project: Joakim Hedegård and Christof Schneider from Swerea KIMAB for valuable discussions and for performing the weld scanning in the experimental part of this thesis, Mattias Clarin from SSAB for providing the steel materials for the test specimens, Tennce Carlsson from Lecor Stålteknik AB for manufacturing the specimens and Paul Lefevre from SONATS for performing the HFMI treatment and the residual stress measurement. Special thanks to Associate Professor Zuheir Barsoum and his co-workers, Doctor Mansoor Khurshid and Ahmad Mahmoudi for performing part of the fatigue experiments at the Royal Institute of Technology in Stockholm.

I also want to thank Caroline, my wife, for all her support and love. I wish you the best in life and a successful Ph.D. defence this winter. Finally, thank you my children, Miryam and Meysam. You are probably not able to read this in a few years' time, but know that you guys are the most important part of my life.

Gothenburg, 2017

Poja Shams-Hakimi

List of publications

This thesis is based on the work contained in the following papers:

Paper I

P. Shams Hakimi, A. Mosiello, K. Kostakakis, and M. Al-Emrani, "Fatigue life improvement of welded bridge details using high frequency mechanical impact (HFMI) treatment," published in *the 13th Nordic Steel Construction Conference*, 2015.

Paper II

P. Shams-Hakimi, H. C. Yıldırım, and M. Al-Emrani, "The thickness effect of welded details improved by high-frequency mechanical impact treatment," published in *International Journal of Fatigue*, vol. 99, Part 1, pp. 111–124, Jun. 2017.

Paper III

P. Shams-Hakimi, F. Zamiri, M. Al-Emrani, and Z. Barsoum, "Experimental study of 40 and 60 mm transverse attachment joints improved by high-frequency mechanical impact treatment," submitted to *Engineering Structures, 2017.*

Author's contributions to jointly published papers

The contribution of the author of this licentiate thesis to the appended papers is described here.

I. Responsible for the major part of writing and planning of the paper. The author performed the literature review, a parametric study and was responsible for the conclusions.

II. Responsible for the major part of writing and planning of the paper. The author performed the major part of the literature review and was responsible for the results, discussions and conclusions.

III. Responsible for the major part of writing and planning of the paper. The author planned and executed major part of the fatigue experiments and was responsible for the literature review, the involved calculations, discussions and conclusions.

Additional publications by the author

Conference Papers

P. Shams Hakimi, H. Yildirim, and M. Al-Emrani, "Size effect on the fatigue of High Frequency Mechanical Impact treated welds," in *IABSE Congress Stockholm 2016; Challenges in Design and Construction of an Innovative and Sustainable Built Environment*, Stockholm, Sweden, 2016, pp. 284–291.

Reports

P. Shams Hakimi and M. Al-Emrani, "Post weld treatment - Implementation on bridges with special focus on HFMI," Chalmers University of Technology, Report 2014:8, 2014.

Contents

Extended Summary

1 Introduction

1.1 Background

Fatigue damage evolves progressively under cyclic stresses and strains at levels lower than the elastic limit of the material and results in cracks that initially can be difficult to detect. Under adverse situations and if not detected, such cracks can develop rapidly and cause sudden and brittle failure in structures. This makes fatigue the most critical failure criterion in the design of steel bridges, especially considering the long prescribed service lives of 80 to 120 years during which bridge elements are subjected to millions of loading cycles. Fatigue damage emanates at sites where geometric changes give rise to stress concentration. Weldments are examples of such sites and constitute the most susceptible parts of bridges to fatigue damage. Despite the disadvantages with regards to fatigue, welding is the predominant joining method in steel bridges.

With the introduction of the Eurocodes as design standards for bridges in Sweden, the fatigue limit state has been found to govern the design to a greater extent and new bridge designs require more material than before. This has led to the loss of competitiveness for steel and composite steel/concrete bridges. Yet, these types of bridges possess advantages such as high strength to weight ratio, the possibility of pre-manufacturing and quick assembly as well as launching methods that are essential in certain circumstances. Utilising existing technologies to solve the fatigue problem would, therefore, imply obvious advantages and potentially reduce the material consumption in steel and composite bridges.

Post-weld fatigue improvement techniques can substantially increase the endurable number of load cycles in structures by improving the fatigue properties of welds in which the fatigue cracks emanate from the weld toe; the transition region between base and weld material. Such techniques include *stress concentration reducing* or *residual stress based* methods. The stress concentration reducing techniques involve grinding or TIG re-melting of the weld toe, improving the fatigue strength through increased transition radius. The residual stress based methods mainly rely on either removing the tensile residual stresses from welding or even introducing compressive residual stresses which delay the formation and development of fatigue cracks. Such techniques include post-weld heat treatment, shot peening, needle/hammer peening or the more recent high-frequency mechanical impact treatment.

High-frequency mechanical impact (HFMI) treatment has proven to be one of the most efficient techniques for fatigue enhancement [1]–[4], yielding the greatest improvement in the high-cycle regime where the fatigue life is governed by low stress magnitudes, such as in bridges. This treatment improves the fatigue strength at the weld toe through high-frequency impacts (>90 Hz) of indenters made of high-strength steel, normally attached to hand-held tools. The improvement mechanisms involve 1) introduction of compressive residual stresses through plastic deformation which reduces the effective part of the stress range causing fatigue damage, 2) smoothening of the weld toe which decreases stress concentration and 3) strain hardening of the material through plastic deformation which increases the material's resistance against crack initiation. HFMI tool manufacturers use brand names such as Ultrasonic Impact Treatment (UIT), Ultrasonic Needle Peening (UNP), Ultrasonic Peening (UP), High-frequency Impact Treatment (HiFIT) and Pneumatic Impact Treatment (PIT). In general, all these HFMI tools produce equivalent improvement of the fatigue strength at the weld toe [5][6]. An example of an HFMI tool and indenters is shown in Figure 1.

Figure 1. An example of HFMI tool and indenters [7].

In 2016, the International Institute of Welding (IIW) adopted the HFMI technique, providing fatigue assessment and quality assurance recommendations [8]. The IIW specifies that the improvement recommendations are valid for the fatigue strength of the weld toe in steel materials with yield strengths ranging from 235 to 960 MPa and plate thicknesses between 5 and 50 mm [8]. The recommendations are based on studies that show a change of slope of the *S-N* curve, m, from 3 to 5 [9], material strength dependence [10] and influence of variable amplitude loading [5][11] on the fatigue strength after HFMI treatment. Furthermore, conditions are given for the stress ratio ($\sigma_{min}/\sigma_{max}$), R, and maximum stress which have an important influence on the fatigue strength after HFMI treatment [6][12]–[14]. The IIW suggests a thickness correction factor to account for the reduced fatigue strength usually observed with increased main plate thickness [8]. This correction is based on fatigue test data of toe ground joints [15].

Although a significant amount of experimental studies are available in the literature on the fatigue strength improvement by HFMI treatment, no studies have investigated the thickness effect exhaustively. The majority of available experimental results are for plate thicknesses less than 30 mm and yield strengths above 400 MPa which do not include typical cases in bridges. In bridge applications, plate thicknesses of up to 40-50 mm are common, extending to over 80 mm in some cases and the most widely used steel grade is S355. On thinner plates ($\leq$ 30 mm), the effect of thickness has been studied in [16]–[21]. Due to rather narrow ranges of thicknesses investigated, often, no thickness effect has been observed. Recently, Iwata et al. [4] studied the thickness effect of non-load-carrying transverse attachment joints of up to 50 mm thickness and observed a pronounced thickness effect. However, this study included few experiments, with seven results on thicknesses above 30 mm. Schaumann et al. [22] also tested non-load-carrying transverse attachment joints, with five test results on 90 mm thick specimens. The obtained fatigue strengths in both [4] and [22] were higher compared to the design S-N curves recommended by the IIW [8].

1.2 Aim and objectives

The overall aim of this thesis is to investigate the fatigue performance of HFMI-treated welds under conditions applicable for bridges. To that end, the following research questions in the form of objectives have been defined for this project:

1) To study the potential benefit of HFMI treatment on common bridge types in terms of material saving and to identify limitations of using HFMI in bridge applications in terms of degree of improvement and steel grades.

2) To study the influence of thickness on the fatigue strength based on existing experimental fatigue results published in the literature of HFMI-treated joints.

3) To investigate the degree of fatigue strength improvement by HFMI for a typical fatigue-prone bridge detail with large main plate thicknesses and relevant material yield strengths through experimental work and analytical models.

1.3 Method and scientific approach

The scientific approach comprises literature reviews, experimental work and the use of analytical models and numerical tools. Based on a literature review, a database was established of published experimental fatigue test results of HFMI-treated joints and the fatigue strength with respect to thickness was analysed statistically. In this thesis, experimental fatigue tests were conducted under four-point bending of non-load-carrying transverse attachment details with large main plate thicknesses which were lacking in the literature. Crack growth analyses with linear elastic fracture mechanics were employed in order to modify the experimental results to represent axial loading.

1.4 Limitations

- This thesis focuses on the improvement by HFMI treatment, providing necessary knowledge for design recommendations and implementation on new bridge designs. Thereby, the scope does not include aspects of repair or retrofit of existing structures.

- The investigations of the effect of thickness on fatigue strength are limited to low mean stress and constant amplitude fatigue loading.

- The experimental work presented in this thesis is performed under constant amplitude loading and low mean stresses, as conventionally done to establish design *S-N* curves. Thus, not providing information related to fatigue under realistic load conditions in bridges.

1.5 Outline of the thesis

This thesis is mainly based on one conference paper and two journal papers. The outline is as follows:

Chapter 1: This chapter gives a background for the motivation of the research and defines the problem statements in the form of objectives.

Chapter 2: The fatigue behaviour of welded joints is described in this chapter followed by a general description of the thickness effect and how the HFMI treatment affects the fatigue behaviour.

Chapter 3: Objective 1 is treated in this chapter which regards fatigue aspects in bridges and the potential benefits and limitations of HFMI application for bridges. Results from Paper **I** in the form of a parametric study are included in this chapter.

Chapter 4: Based on a literature review performed in Paper **II** and an established database, the thickness effect in HFMI-treated joints is assessed in this chapter, fulfilling objective 2.

Chapter 5: Experimental results from Paper **III** are presented in a condensed format in this chapter in fulfilment of objective 3.

Chapter 6: The main conclusions of the three papers are summarised in this chapter followed by suggestions for further research.

2 Fatigue of welded joints

2.1 General

Fatigue is a damage process that occurs progressively in metallic components and structures which are subjected to repeated stress cycles. The damage process can start from the very first cycle and for stresses far below the elastic limit of the material. This process involves several stages, however, a common simplification is to divide the total fatigue life into fatigue (macro) crack initiation and fatigue (macro) crack propagation until failure [23].

The position of final fatigue failure is decided by the weakest link in the material. In a plain metallic plate, the weakest link is often an inhomogeneity on the surface or at the plate edge. Surface roughness is an important factor, therefore, the fatigue strength can be influenced by the method of cutting. In a component without notches, the number of cycles to initiate a macro-crack dominates the fatigue life, and the fatigue limit, below which macro-cracks do not initiate, is approximately one-third of the ultimate strength [24], $0.3f_u$. Due to the absence of notches, the macro-crack propagation rate is relatively low. In notched components with holes or sharp edges, the position of fatigue failure is normally at the site of highest stress concentration. Still, the fatigue crack initiation can constitute a significant portion of the fatigue life, however, the fatigue limit is reduced to approximately $0.3f_u/K_t$, where K_t is a factor by which the nominal stresses are magnified at the notch. The crack propagation rate is faster than in plain components due to the presence of stress concentration. In welded components, the transition between weld and base material is the weakest links, since, in addition to high stress concentration, tensile residual stresses are present from the cooling process after welding. In addition, weld defects such as undercuts, porosities, inclusions or spatter can exist in the weld toe region, acting as sharp notches. A consequence of welding is a dramatic decrement in fatigue strength, as illustrated in Figure 2. Because the fatigue life of welded components is mainly governed by crack propagation, the second consequence of welding is that the fatigue strength becomes relatively independent of the material strength, see Figure 3 for sharp notches. The growth rate is relatively high due to significant notch effect and tensile residual stresses.

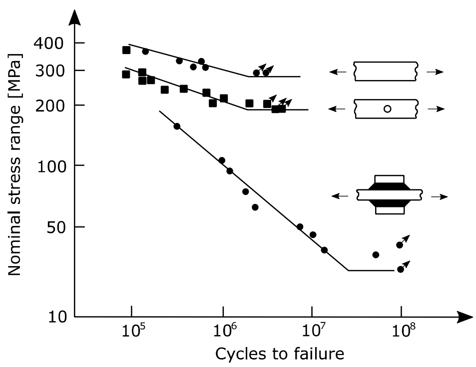

Figure 2. Fatigue strength decrement due to notches and welds according to [25].

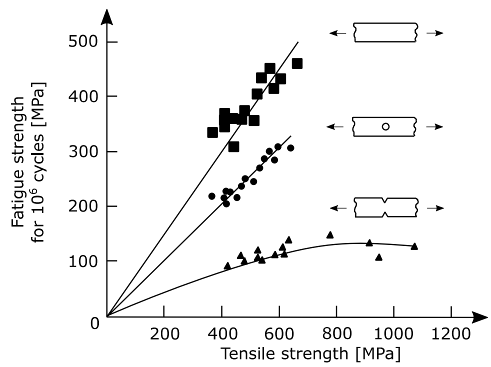

Figure 3. Fatigue strength dependence on material strength according to [26].

2.2 Influence of thickness

The thickness effect entails fatigue strength reduction for an increase of load carrying plate thickness. The thickness effect is part of a more general term called the size effect in which additional geometric quantities that influence the fatigue strength are included. The size effect can be divided into three categories: the statistical, technological and geometric size effects [27].

- The statistical size effect reflects a decrement of fatigue strength due to a higher probability of the existence of defects in a larger material volume or component. In welded joints, the statistical size effect is governed by the weld length since fatigue cracking usually starts from the weld toe [28].
- The technological size effect regards the effect of size during the manufacturing process. Commonly, welding of larger components induces higher tensile residual stresses which reduce the fatigue strength. In addition, thicker plates may have coarser grains and lower yield strengths [29].
- The geometric size effect includes the influence of geometry on the stress state during loading. Taking a non-load-carrying transverse attachment detail as an example, the influencing geometric sizes on the fatigue strength are main plate thickness (t_1), weld size (l) and attachment plate thickness (t_2), see Figure 6. All of these dimensions increase the stress concentration at the weld toe if enlarged, thereby shortening the fatigue crack initiation life, or in the case of welded joints, resulting in faster crack growth during the early propagation phase [28][30]. Aside from increasing the stress concentration, an increased main plate thickness also reduces the rate of which stresses decrease through the thickness, i.e. the stress gradient, Figure 6a. For thicker main plates, a lower stress gradient results in larger stresses near the surface of the weld toe which shortens the crack propagation life. In addition, a similar stress gradient effect arises when a component is subjected to bending, which manifests in a more severe thickness effect.

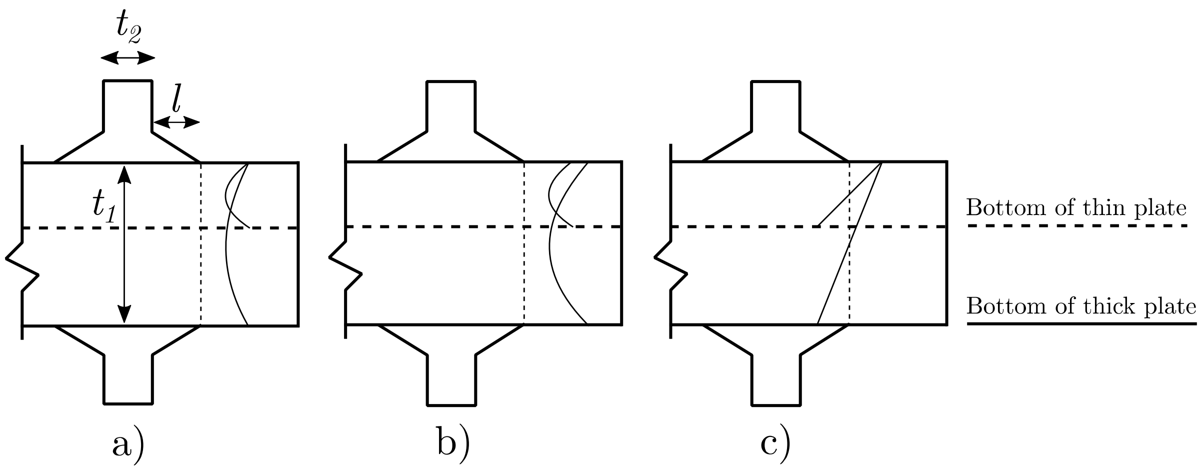

Figure 6. Geometric thickness effect due to a) stress gradient and b) stress concentration. c) Effect of bending.

Gurney's approach [31] is a well-established approach for treating the thickness effect and has been adopted in several design codes and guidelines [28]. With this approach, fatigue test results can be evaluated in terms of stress-thickness (*S*-*t*) curves. Equation (1) describes the function of the *S*-*t* curve. The fatigue strength, ΔS, corresponding to a thickness, t, can be found from the *S*-*t* curve which is defined by a reference fatigue strength, ΔS_{ref}, and thickness, t_{ref}. The exponent, n, also referred to as thickness correction exponent describes the slope of the *S*-*t* curve and is a measure of the severity of the thickness effect. With this approach, Equation (1) can be used to reduce the fatigue strength of details with plate thicknesses greater than t_{ref}. The value of t_{ref} is often set to 25 mm in design codes [28].

$$\Delta S = \Delta S_{ref} \left(\frac{t_{ref}{}^{n}}{t} \right) \tag{1}$$

Paper **II** in this thesis elaborates more on the thickness effect through a literature review and provides new information about the thickness effect of different types of welded joints improved by HFMI treatment. Chapter 4 gives a summary of the results of Paper **II**.

2.3 Influence of HFMI treatment

In principle, HFMI treatment of welded joints results in similar fatigue behaviour as that of notched specimens without welds, as shown in Figure 2 and Figure 3. The *S*-*N* curve of HFMI-improved joints is situated at higher stress ranges, typically with a shallower slope and higher fatigue limit than for as-welded joints. In addition, the fatigue strength of HFMI-treated joints is dependent on the material strength. The greatest improvement is obtained in high-cycle fatigue loading whereas, in low cycle fatigue, the difference in fatigue strength compared to the as-welded state might be negligible, see e.g. [12].

As oppose to as-welded joints, the fatigue strength of HFMI-treated joints is dependent on both the mean and maximum level of the applied load. The mean level is usually represented by the stress ratio, R, and influences the portion of the stress range that causes fatigue damage. Since part of the stress range is consumed by the induced compressive residual stresses, a higher mean stress causes a larger part of the stress range to be tensile and damaging. The maximum load level requires restriction due to the concentration of stresses at the weld toe which increases the risk of local yielding, even for nominally elastic stresses. Yielding can cause relaxation of the compressive residual stresses, thus, reducing or even completely removing the benefit of HFMI treatment [32][33].

3 Fatigue improvement of bridges

In the design of bridges, several criteria regarding the functionality and capacity of the structure expressed as limit states must be considered. The ultimate limit state (ULS) covers the resistance against exceptionally heavy short-term loads such as truck-loads or dynamic effects from high-speed train passages. In the ULS, the structural integrity is permanently lost which is referred to as a structural failure. Failure modes in this limit state include yielding of the material or instability phenomena such as local or global buckling. The fatigue limit state (FLS) is closely related to the ULS in terms of permanent loss of structural integrity if fatigue cracks arise. However, the fatigue process occurs over time and for much lower loads than the ULS, thus, a separate limit state for fatigue is appropriate. Lastly, the serviceability limit state (SLS) must be considered, involving anything that may compromise the functionality of the bridge without affecting the structural integrity. Vertical deflection is included in this category. Aside from the aesthetical aspect and passenger comfort, the limitation of vertical deflection is of importance for safe traffic passage under the bridge and, in the case of railway bridges, for the proper functionality of the rails. The restriction of vertical deflection is, therefore, greater for railway bridges.

The most common types of steel bridges are comprised of two or more welded I-beams (girders) as the main load carrying members. Depending on the type of traffic the bridge is intended for and possible requirements on the self-weight, the deck system can be made of either steel or concrete and the bridge can be made continuous over several supports or simply supported. Steel railway bridges are often made simply supported with steel plates as decks with stiffening members underneath the rails. This makes steel railway bridges light compared to the heavy train loads, resulting in relatively low stress ratios, R. Steel railway bridges are the most fatigue sensitive, especially those with short spans. The reason is that shorter spanned bridges experience more damaging cycles per train passage globally. Furthermore, railway traffic is relatively regular in terms of load pattern, therefore, unforeseen overloads are less likely to occur compared to road traffic bridges.

The simplest steel road bridges are made with concrete decks in composite action with steel I-beams. Consequently, such bridges are relatively heavy in comparison to the road traffic and the stress ratios are high. The load variations are also more stochastic compared to railway traffic due to variations in the lateral positions of the traffic and the weights of heavy trucks which in some cases are excessively loaded. This increases the risk of overloads. Moreover, during the construction phase, the steel girders must withstand the weight of the wet concrete which does not contribute to the load carrying capacity. During this stage, lateral torsional buckling of the top flanges is critical for which more detailing in the form of welded cross bracings are necessary, adding to the number of fatigue critical weldments. Additionally, if the bridge spans over several supports, it is normally made continuous which requires additional on-site welding to join smaller bridge segments together.

While the fatigue-prone details in bridges can be relatively limited in number, they require a significant amount of extra material in the flanges compared to what is needed in the ULS, in order to reduce stresses and endure the long service lives. For short-spanned bridges, the critical details with respect to fatigue are usually the welds joining vertical web stiffeners and flanges, see Figure 7a. Eurocode 3 provides a fatigue strength of FAT 71-80 for this detail [34]. For longer bridge spans, butt welding of bridge segments on-site may become necessary, FAT 80-90, sometimes in conjunction with cope-holes, FAT 71. With HFMI treatment of these details, the flanges can be made thinner, resulting in significant weight reduction. The use of high-strength steels in conjunction with HFMI treatment can allow further material savings, however, vertical deflection or instability phenomena can become restricting. Moreover, the maximum fatigue strength improvement may be limited by the strength of the longitudinal flange-to-web welds, FAT 125, since these welds can be too long to post-weld treat and fatigue cracks may start from inner weld defects, e.g. porosities.

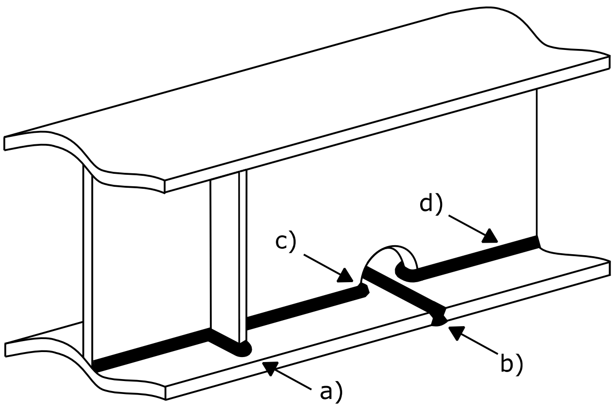

Figure 7. Typical weldments in bridges. a) non-load-carrying transverse attachment, b) butt weld, c) cope-hole and d) longitudinal flange-to-web weld.

3.1 Parametric study

Although fatigue sensitive, fortunately, railway bridges are the most suitable for HFMI treatment due to low stress ratios. A parametric study was performed in this thesis in order to obtain a preliminary quantification of the potential material saving following post-weld treatment of simply supported railway bridges with spans ranging between $10 \leq L \leq 30$ meters. The bridges were verified for ULS, FLS and SLS with the Eurocode train load model LM71 [35]. The ULS checks included calculations for bending, shear and bending-shear interaction including local buckling. The FLS check was conducted for heavy traffic mix of trains with a partial safety factor of $\gamma_{Mf} = 1.35$ and a service life of 100 years, with the linear damage accumulation method. A transverse stiffener in the mid-span with detail category FAT 71 was assumed in the lower flanges as the fatigue critical detail. Fatigue improvement was conservatively considered with an increase of three FAT-classes, resulting in FAT 100, with the slope of the S-N curve equal to $m = 3$. For the SLS condition, a limitation of vertical deflection of $L/800$ was used. Figure 8 displays a typical cross section of a railway bridge included in this study.

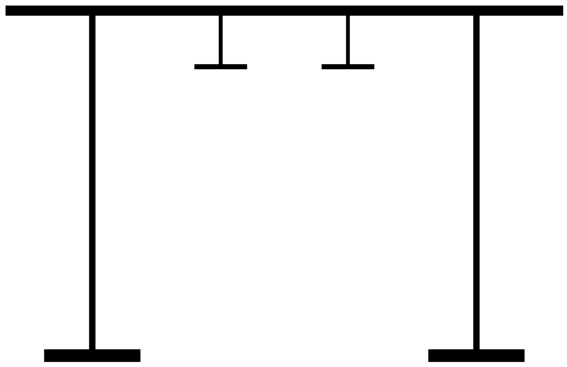

Figure 8. A typical cross section of a railway bridge included in the parametric study.

Initial bridge designs were produced with a steel grade of S355 (minimum yield strength of 355 MPa) without post-weld treatment, referred to as *Original designs*. The flange thicknesses were kept constant, equal to 35 and 45 mm for the upper and lower flanges respectively, while the web heights varied to meet the design requirements for the different span lengths. After application of post-weld treatment and implementation of higher steel grade, S460, the upper and lower flange thicknesses could be reduced to 20 and 30 mm, respectively, including minor changes of the bottom flange widths. In Figure 9, the utilisation ratios (UR) for the different limit states are presented as a function of span length for each step of the calculations. The UR is defined as the ratio of load effect to resistance such that a UR < 1.0 implies fulfilment of the requirements of the limit state. From Figure 9a, it is apparent that FLS is most dominant for shorter span lengths. As can be seen in Figure 9b, a significant decrease of the UR of FLS occurs after post-weld treatment and ULS becomes the dominant design criterion. Increasing the material strength to S460 lowers the UR of ULS to the same level as SLS, indicating that greater material strengths are unnecessary, see Figure 9c. Finally, the UR's of the new designs with reduced flange thicknesses are depicted in Figure 9d, showing that there remains more potential for material saving for the shorter spans if a greater fatigue strength improvement or a greater slope is assumed, e.g. $m = 5$. Table 1 summarises the percentage material saving in the beams as a result of post-weld treatment and the use of higher steel strength.

Table 1. Material reduction of bridge cross sections in the parametric study.

L = 10 m	***L = 15 m***	***L = 20 m***	***L = 25 m***	***L = 30 m***
31%	26%	25%	23%	21%

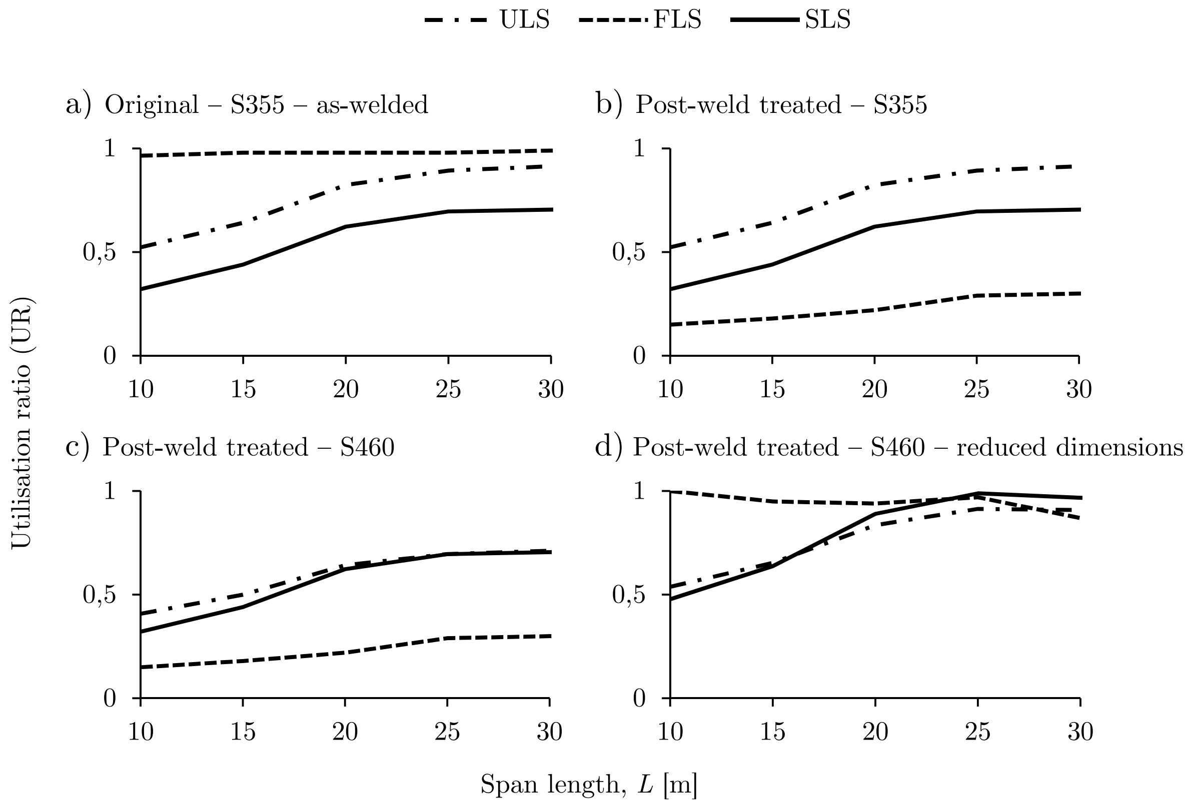

Figure 9. Utilisation ratios for different limit states in the parametric study of simply supported railway bridges.

3.2 Case studies

In addition to the parametric study, case studies were performed on three different bridges that have been built in Sweden without post-weld treatment. They consisted of a simply supported railway bridge and two road bridges, one simply supported and one continuous, all of which were designed according to the Eurocode standards with the lambda method for fatigue stress assessment [35]. The bridges were re-designed with smaller flange thicknesses after accounting for post-weld treatment and increased material strength where necessary, whereafter, the material savings were assessed. Unlike the parametric study, the obtained material savings here were more accurate since the existing design calculations were re-used. Consequently, the calculated material savings were more representative and isolated to only the effect of the fatigue strength improvement and the potential use of high-strength steels.

Case 1: Simply supported Railway Bridge

This railway bridge was comprised of 15 identical 10 m long segments, each made of two I-beams with a common top flange as the deck. The original flange thicknesses were 30 mm at the top and 45 mm in the bottom, with a bottom flange width of 600 mm, see Figure 10. The steel grade was S355 and the fatigue critical detail was a non-load-carrying transverse attachment joint in the mid-span, FAT 80. An increase of three FAT-classes, retaining the same steel grade, allowed for thickness reduction in the flanges which resulted in 17% material saving in the beams, corresponding to 30 tonnes of steel for the entire bridge. The treatment length was estimated to about 30 m in total. A stress ratio of approximately $R = 0.08$ was calculated and the maximum stress ranges were 59 MPa in the original and 78 MPa in the new designs. Although an improvement of more than three FAT-classes could be achieved, vertical deflection would not have allowed for more material reduction in this case. The utilisation ratios for the design limit states are summarised in Table 2.

Figure 10. Railway Bridge during construction, case 1. Original flange dimensions and the dimensions after HFMI treatment in parenthesis. Red lines indicate HFMI-treated areas.

Table 2. The utilisation ratios for the original design and the new design with post-weld treatment.

Utilisation	***ULS***	***FLS***	***SLS***
Original	0.63	1.00	0.82
New	0.79	0.96	1.00

Case 2: Simply supported Road Bridge

This simply supported road bridge spanned 32 m and consisted of three different segments of steel I-beams with various flange dimensions for the purpose of optimal material use. The top flange thicknesses were 20 mm in the two outer segments and 25 mm in the middle one, whereas the bottom flanges were made of 25 mm in the outer and 32 mm in the middle. The deck was made of concrete which acted compositely with two I-beams through shear studs. A steel grade of S355 was used in the top flanges and webs, whereas an S420 steel was used in the bottom flanges. The ultimate and fatigue limit states governed equally in this design. Due to the heavy concrete deck, high stress ratio of at least $R = 0.65$ were calculated for the critical details.

The calculations for post-weld treatment was performed with two alternatives. For both alternatives, the top flange thicknesses could not be reduced due to limited lateral torsional buckling capacity during the construction phase, thus, the material saving was only a result of bottom flange thickness reduction. For alternative 1, only a transverse non-load-carrying attachment in the mid-span was post-weld treated, an approximate treatment length of 2 m. Thereby, butt welds that joined the three bridge segments were not treated. Decreasing the bottom flange thicknesses in the middle segment to 15 mm and increasing the yield strength to 620 MPa resulted in a material saving of 10%, corresponding to 2.5 tonnes of steel. This assumed a FAT-class increase from FAT 80 to FAT 125. In alternative 2, the butt welds were treated in addition to the transverse attachments, assuming the same FAT-classes and a total treatment length of 16 m. A material saving of 22% in the beams was realised by reducing the bottom flange thicknesses to 15 and 11 mm in the middle and outer segments, respectively, and increasing the yield strength to 620 MPa. This corresponded to 5.4 tonnes of steel saving. The maximum stress range in the original design was 54 MPa while it increased to 90 MPa after treatment and thickness reductions. More details on this study are available in [36].

Figure 11. Composite Road Bridge, case 2.

Table 3. The utilisation ratios for the original design and the new designs with post-weld treatment.

Utilisation	***ULS***	***FLS***	***SLS***
Original	0.89	0.89	0.54
New, Alt. 1	0.97	0.96	0.78
New, Alt. 2	0.98	0.97	0.85

Case 3: Continuous Road Bridge

The last bridge was a continuously supported composite road bridge with five spans and a total length of 130 m, see Figure 12. Similar to the simply supported road bridge, this bridge consisted of two I-beams with varying flange thicknesses in composite action with a concrete deck. The stress ratio was somewhat lower in this bridge, with $R = 0.52$, and the steel grade was S355 in all parts. Around 40 critical details in the form of transverse attachments from vertical stiffeners and cross bracings as well as in-situ and workshop butt welds required post-weld treatment. The total treatment length was approximated to 154 m for the whole bridge. The welded details were all FAT 80 in the as-welded state and were assumed to FAT 125 after treatment. The steel grade was increased to S460 for the new design. Typical examples of some thickness reductions were 40 to 30 mm in the bottom and 30 to 20 mm in the top flanges over the supports. The top flanges were kept relatively unchanged in the mid-spans due to the risk of buckling in the construction phase, whereas the bottom flange thicknesses were typically reduced from 40 to 20 mm. The maximum stress range in the original design was 44 MPa and increased to 66 MPa after treatment and thickness reductions. As a simplification, the bending moment distribution of the original design was re-used in the new design. In total, 23% material was saved in the beams which corresponds to 30 tonnes of steel. Table 4 shows the maximum utilisation ratios.

Figure 12. Composite Road Bridge during construction, case 3.

Table 4. The utilisation ratios for the original design and the new design with post-weld treatment.

Utilisation	***ULS***	***FLS***	***SLS***
Original, support	1.00	0.41	-
Original, span	0.87	0.96	0.34
New, support	1.00	0.51	-
New, span	0.99	0.98	0.47

3.3 Summary of bridge investigations

Based on the studies in this chapter, conclusions can be drawn regarding the fatigue design of some commonly built bridge types in Sweden today and the feasibility of post-weld treatment in these contexts. The conclusions may not be applicable for other bridge types than those investigated herein, such as truss bridges where the static systems are significantly different or road bridges with orthotropic steel decks where complex weldment are required. With this in mind, the following conclusions can be drawn:

- A material saving of around 20% can be achieved in the main load-carrying members of bridges through post-weld treatment and potentially the use of higher strength steel, depending on the type of bridge and circumstances of the specific case.

- When bridges are made lighter by using thinner plates, the fatigue strength reduction due to thickness effect diminishes. In addition, HFMI-treated welds possess milder thickness effect than what is recommended for as-welded joints, as described in chapter 4, which implies further efficiency.

- It is apparent that transverse non-load-carrying attachments and butt welds usually make up the fatigue critical details in the investigated bridge types. In these details, cracking occurs at the weld toe. However if treated, root failure might become a competing failure mode to the treated weld toe. One way of solving this issue is to use full-penetration welds.

- The stress ranges are typically around 40 to 60 MPa in fatigue loading in bridges, increasing up to 90 MPa if the designs are optimised with post-weld treatment. This puts the fatigue behaviour in the high-cycle regime where the improvement by HFMI treatment is the greatest.

- The stress ratio can be significantly different depending on bridge type. Bridges with low self-weight compared to the traffic load, such as steel railway bridges, have low stress ratios of around $R = 0.1$, while composite bridges with concrete decks may have much higher stress ratios of up to $R = 0.7$. These values were found by the lambda method [35]. Thus, if the damage accumulation method with stress blocks was used instead, or if the actual cycle-by-cycle stress ratio from variable amplitude loading were to be assessed, the values might become different. Although the effectiveness of HFMI treatment diminishes with increasing stress ratio, a number of investigations have shown that significant improvement is gained even for high stress ratios if peening treatment is performed under tensile load [37]–[40].

- It is recalled that the use of high-strength steel can be restricted by vertical deflection in the case of railway bridges, and instability phenomena such as global buckling during the construction phase of composite road bridges. Based on the studies above, a steel grade of S460 seems appropriate in conjunction with HFMI treatment on bridges.

- Very long welds such as the longitudinal flange-to-web welds might not be feasible to post-weld treat. Fatigue cracks in such welds may start from inner weld defects which could remain unaffected after treatment by HFMI. These welds can have a FAT-class of 125 at best, thus, restricting the fatigue strength improvement for other weldments on the flanges.

4 Thickness effect in HFMI-treated joints

The International Institute of Welding (IIW) recommends a thickness correction exponent of $n = 0.2$ for joints improved by any post-weld treatment method, e.g. grinding, TIG re-melting or the peening methods [8][41]. For joints in the as-welded state, the recommended exponent is $n = 0.3$ for transverse and longitudinal attachments and $n = 0.2$ for transverse butt welds [15].

In Paper **II**, a review of thickness effect studies on as-welded joints was performed in order to better understand by which mechanisms HFMI treatment could influence the thickness effect. The review indicated that the thickness effect of butt welded joints is technological since the thickness effect could be eliminated by both stress relieving and high mean stress loading according to [42][43]. For as-welded longitudinal attachment joints, most of the reviewed studies indicated that the thickness effect is negligible, with thickness correction exponents close to zero. Fracture mechanics calculations performed by Gurney [31] even showed a "reverse" thickness effect (negative n) in longitudinal attachment joints and it was suggested not to reduce the fatigue strength due to thickness increase. Studies on transverse attachment joints showed thickness correction exponents between $n = 0.2$ and 0.3 for proportional scaling, i.e. proportional size increase of attachment and weld with respect to main plate thickness. In cases where attachment and weld sizes are kept constant, increasing the main plate thickness results in a thickness correction exponent of $n = 0.05$-0.1 [44]–[46].

4.1 Database

A database comprising experimental fatigue test results for HFMI-treated joints was established to evaluate the thickness effect in these joints. The database included 582 fatigue test results of small-scale specimens from 28 different studies. The specimens included transverse butt joints and joints with non-load-carrying transverse and longitudinal attachments which were loaded in axial tension under stress ratios of $0 \leq R \leq 0.1$. Runouts and results above 5 million cycles were excluded as well as failure modes which were explicitly reported elsewhere than the weld toe. Table 5 gives an overview of the database.

Table 5. Experimental studies on HFMI-treated joints included in the database for thickness effect evaluation.

Ref	***Authors***	***Detail***	***t [mm]***	***k***
[47]	Abdullah et al., 2012	B	5	2
[48]	Hrabowski et al., 2014	B	8	8
[49]	Huo et al., 2000	B	8	8
[50]	Janosch et al., 1996	B	9.5	8
[51]	Kuhlmann and Günther, 2009	B, C	12	3, 14
[52]	Leitner et al., 2014	B, T, L*	5	28, 31, 32
[18]	Ummenhofer et al., 2006	B, B, T	8, 30, 30	8, 7, 6
[6]	Ummenhofer et al., 2011	B, L, L	30, 16, 30	46, 32, 30
[12]	Wang et al., 2009	B, L	8	11, 8

Table 5. Continued.

[53]	Weich, 2008	B, L	16	104, 28
[17]	Deguchi et al., 2012	C, L	16	2, 3
[54]	Ermolaeva and Hermans, 2014	C	20	8
[55]	Han et al., 2009	C	16	7
[4]	Iwata et al., 2015	C	10, 22, 40, 50	3, 3, 4, 3
[56]	Kuhlmann et al., 2005	C	12	12
[57]	Kuhlmann et al., 2006	C	12	3
[58]	Okawa et al., 2012	C	20	3
[59]	Tehrani Yekta, 2012	C	9.5	13
[60]	Zhao et al., 2016	C	10	5
[61]	Haagensen et al., 1998	L	6	4
[62]	Haagensen and Alnes, 2005	L	8	15
[63]	Huo et al., 2005	L	8	4
[64]	Lihavainen and Marquis, 2004	L*	8	10
[65]	Lihavainen et al., 2004	L*	5	5
[66]	Marquis and Björk, 2008	L	8	5
[67]	Martinez et al., 1997	L	12	12
[21]	Vanrostenberghe et al., 2015	L	5, 10, 20	12, 21, 5
[68]	Wu and Wang, 2012	L	8	6

*One-sided attachment. B = butt weld, C = cruciform joint, T = T-joint, L = longitudinal attachment, t = main plate thickness, k = number of specimens

4.2 Thickness correction exponents

Prior to thickness evaluation, the collected data had to be adjusted for fatigue strength variations caused by different yield strengths. An approach was developed for this purpose such that all data could be adjusted to represent a yield strength of 355 MPa, more details are given in Paper **II**. Figure 13 shows the obtained S-t relationships together with the thickness correction exponents for the three investigated details. The ΔS_m value for each data point represents its strength at two million cycles assuming a slope of $m = 5$. Although the experiments were conducted with low mean stresses, the butt welded joints showed a weak thickness effect with $n = 0.055$, suggesting that peening eliminates the presumed technological thickness effect observed in the as-welded state. For the two remaining details, a statistical method called *multiple regression with dummy variables* was utilised (see Paper **II**) in order to obtain a common thickness correction exponent for groups where the detail geometries differed significantly. In here, different groups were established for double (cruciform) and single sided (T-joints) transverse attachments and for various lengths of longitudinal attachments, L. An exponent of $n = 0.207$ was obtained for transverse attachment joints, close to the thickness correction exponent recommended by the IIW. For longitudinal attachments, a relatively strong reverse thickness effect was observed with $n = -0.188$. It was concluded in Paper **II** that thickness correction should not be made for longitudinal attachments and that the current recommendation of $n = 0.2$ can be over-conservative for butt welded joints. Overall, HFMI-treated joints show a weaker thickness effect than what is recommended for as-welded joints resulting in an additional source of improvement for thicker plates.

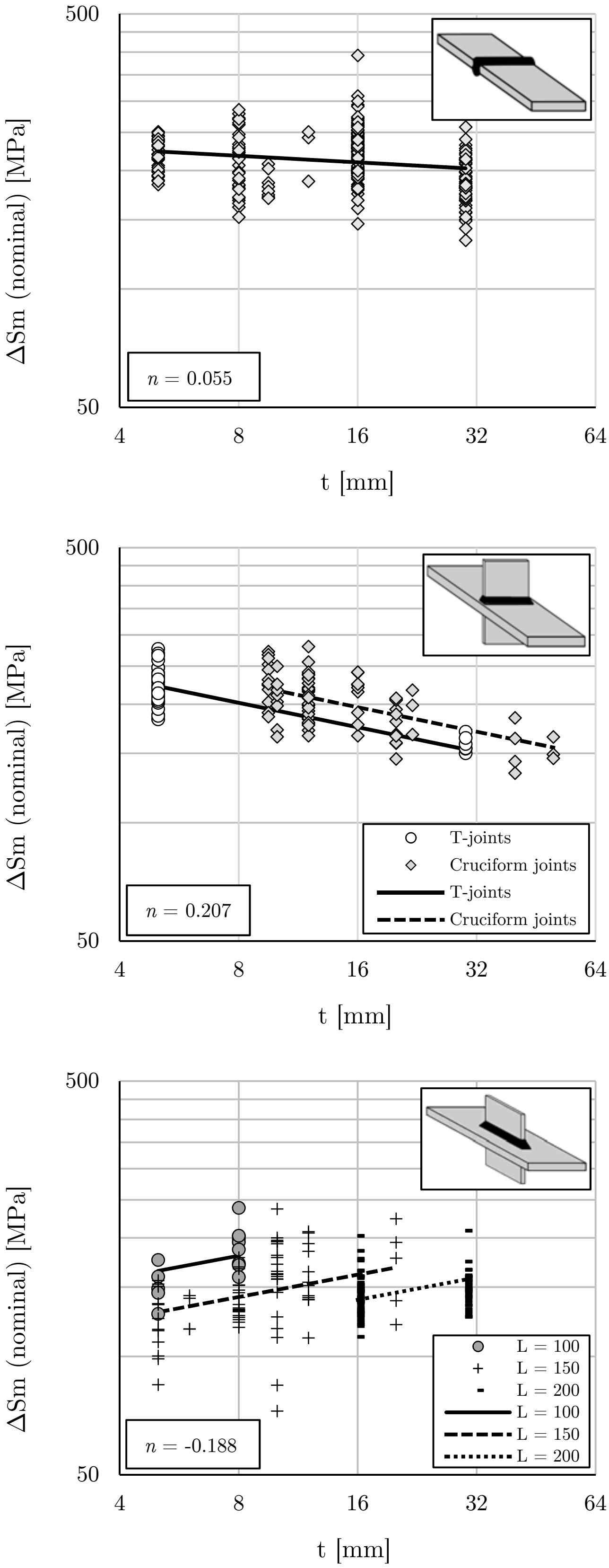

Figure 13. Thickness effect of HFMI-treated joints.

5 Experimental investigation

Fatigue experiments were conducted on 40 and 60 mm non-load-carrying transverse attachment joints under four-point bending, both in as-welded and HFMI-treated states. In addition to fatigue experiments, weld toe scanning, residual stress measurements and microstructural investigations were performed. Fracture mechanics calculations were used to investigate the corresponding fatigue strengths under axial loading. This chapter presents a summary of the investigations and results. For detailed information, see Paper **III**.

5.1 Overview

A total of six test series were produced for fatigue experiments, two of as-welded and four of HFMI-treated specimens, see Table 6. Due to availability in the inventory of SSAB, the delivered steel plates varied somewhat between different series. For instance, a thickness of 38 mm was delivered for series 1 and a steel grade of S500 for series 2. The S500 steel had similar mechanical properties as the S460 steel of the same thickness. The specimen geometry and loading arrangement is shown in Figure 14. The specimen geometries were chosen to represent a typical weldment in bridges; a vertical web stiffener welded to a flange.

The welding was performed by Lecor Stålteknik AB with flux-cored arc welding and the same welding procedures were performed for all series. The welds were made with full penetration to avoid root failure. The specimens to be improved by HFMI treatment were ground prior to peening to achieve a weld quality class B according to ISO 5817 [69]. The as-welded specimens in series 1 and 2, however, were not ground. All specimens were welded in an upright position, resulting in a smooth transition from the main plate to weld metal, see Figure 14. HFMI treatment was performed with a single indenter of radius 3 mm with an average treatment speed of 200-300 mm/min.

Table 6. Test series and mechanical properties. Number of specimens indicated with k.

Series	*Specimens*	*t [mm]*	*Steel grade*	f_y *[MPa]*	f_u *[MPa]*	*Elongation [%]*	*k*
1	460-38-AW	38	S460M	562	659	22	7
2	500-60-AW	60	S500G2M	471	596	25	6
3	355-40-HFMI	40	S355K2+N	382	531	25	10
4	460-40-HFMI	40	S460M	566	639	23	11
5	355-60-HFMI	60	S355K2+N	361	532	25	10
6	460-60-HFMI	60	S460G2M Z35	494	597	26	10

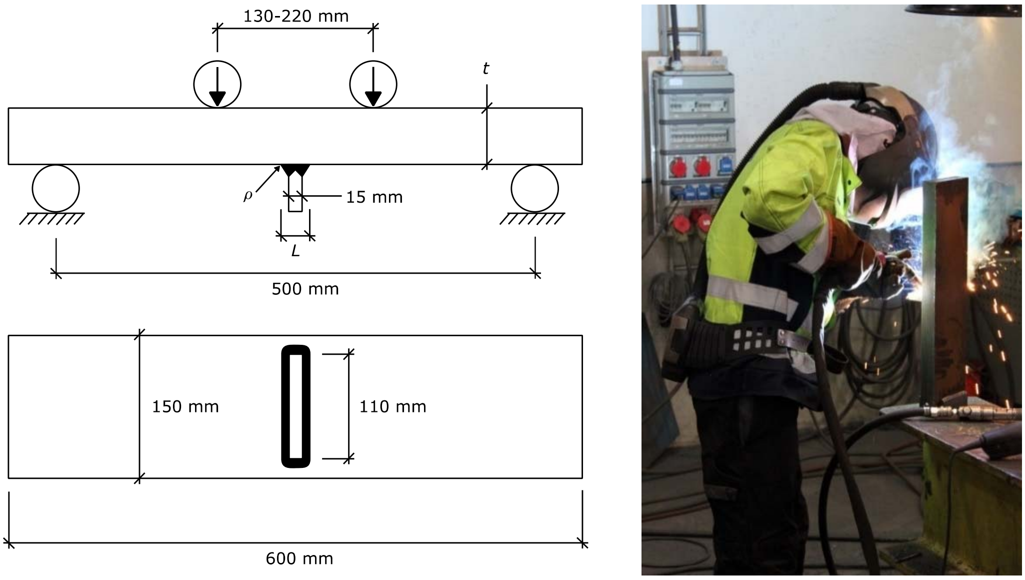

Figure 14. Test specimen dimensions and loading arrangement and picture of the manufacturing.

5.2 Results

Microstructural investigation

The microstructural investigation was performed with optical microscopy with the largest magnification of 50 times. Two HFMI-treated specimens were included of steel grades S355K2+N and S460M. It was observed that in the two investigated samples, HFMI treatment was predominantly performed on the weld materials, causing some degree of observable grain elongation through plastic deformation. However, better images were considered necessary for accurate conclusions. In the regions unaffected by peening, the weld microstructures were similar for the two steel grades. The base materials were of ferritic-pearlitic microstructure in the S355K2+N steel whereas the S460M steel had significantly smaller grains with a bainitic microstructure. Further observations were that the treated surfaces appeared smoother than the base material surfaces, indicating that the surface roughness could have become improved by the treatment.

Weld scanning

The weld scanning was performed by Swerea KIMAB with a laser measurement device. Hundreds of two-dimensional profiles were produced along each weld of all specimens, with a spacing less than 1 mm. An algorithm developed by Stenberg et al. [70] was then used to quantify the weld toe radii and weld toe distances. Due to the upright position of the specimens during welding, large toe radii were produced for the as-welded series. The weld toe radii of all the HFMI series were less than the as-welded series, about the same radius as the pin radius, 3 mm. Furthermore, the radii of series 3 were larger compared to the other HFMI-series, presumably due to over-treatment. The weld toe distances were approximately twice the attachment thickness in all cases.

Residual stress measurements

Residual stress measurements were performed by the company SONATS on four different specimens with 40 mm main plate thickness and two different steel grades, S355 and S460. Measurements of stresses in the weld toes, in the transverse direction to the weld line, revealed zero or compressive stresses for non-treatment (ground) welds and significantly compressive stresses for HFMI-treated welds. The transverse residual stresses became approximately -250 to -300 MPa at the surface, gradually decreasing to zero at a depth of about 1500 μm. The difference between residual stresses for the two steel grades was relatively small, however, more compressive for the S460 specimen in the depths of 200-900 μm in the transverse direction. These results should be treated qualitatively since no corrections were made for surface removal, which could have relaxed the stresses.

Fatigue test results

Fatigue testing was performed with a target stress ratio of $R = 0.1$, although, some variations occurred between the tests within $0.01 \leq R \leq 0.21$, see Appendix of Paper **III**. The loading frequency varied depending on load level, between 3-20 Hz. For the HFMI series, 27 out of 41 specimens failed from the weld toe, with the failure definition as a half thickness crack. The rest were runouts. All 13 as-welded specimens failed from the weld toe. Figure 15 shows the obtained fatigue strengths in terms of characteristic 95% *S-N* curves, also including the characteristic design curves for as-welded (FAT 80) and HFMI-treated (FAT 140) cases. The FAT 140 is chosen here according to the IIW recommendations for HFMI-treated joints with yield strengths between 355 and 550 MPa [8]. No data points fell below the corresponding characteristic design curves.

The obtained fatigue strengths were considerably higher compared to characteristic design curves, also for the as-welded series. The fatigue strengths of the as-welded series were exceptionally high, especially when considering the mean values and natural slopes. This is probably due to the upright positioning during manufacturing which resulted in large toe radii. The low residual stresses as indicated by the residual stress measurements could also have contributed to the high fatigue strengths of the as-welded series. However, the residual stress measurements were performed on ground specimens prior to HFMI treatment, where grinding could have affected the residual stresses to become different from an as-welded specimen.

Evaluation of the thickness effect of the HFMI-treated specimens was performed by implementing a yield strength adjustment method which was developed in Paper **II**, subsequently, deriving n for all HFMI specimens in one group, see Paper **III**. A thickness correction exponent of $n = 0.135$ was obtained, which lies close to reported values of $n = 0.1$ for as-welded joints with constant attachment and weld sizes. The difference may qualitatively be explained by the effect of bending which can result in stronger thickness effect.

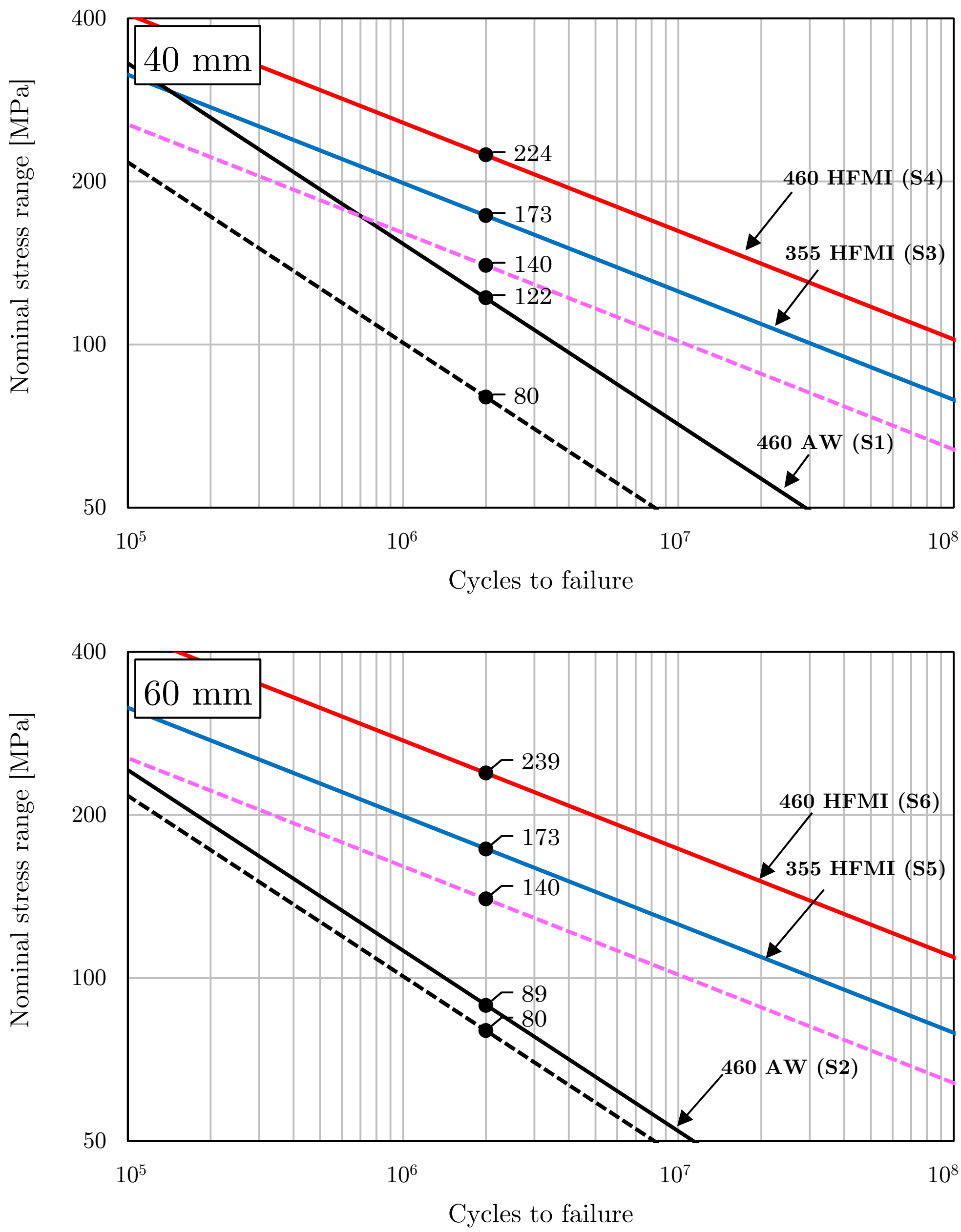

Figure 15. Characteristic 95% S-N curves of all test series with fixed slopes of $m = 5$ for HFMI-treated and $m = 3$ for as-welded specimens.

5.3 Crack growth analyses

Because the recommended FAT-classes for HFMI joints are based on axially tested specimens and the obtained results were situated much higher above the design curves, linear elastic fracture mechanics (LEFM) calculations were performed in order to assess the difference in crack growth between bending and axial loading. With the help of this assessment, the obtained experimental fatigue strengths could be modified to represent axial loading.

To calculate the number of cycles in the crack propagation phase, N_p, Equation (2) according to Paris' law was used, where the range of the stress intensity factor, ΔK, was replaced with ΔK_{eff} to account for crack closure and residual stress effects. The material constants C (N, mm), and m_{LEFM} were taken as $1.8\cdot10^{-13}$ and 3, respectively, in accordance with the IIW document XIII-2370r1-11-XV-1376r1-11 which is an update of the LEFM chapters in [15]. The initial crack depth was chosen to $a_0 = 0.1$ mm and the final crack depth at failure, a_f, to half of the main plate thickness. More elaboration on the choice of a_0 is made in Paper **III**.

$$N_p = \int_{a_0}^{a_f} \frac{da}{C\Delta {K_{eff}}^{m_{LEFM}}} \tag{2}$$

The effective range of the stress intensity factor, ΔK_{eff}, was calculated according to Equation(3), as suggested by Bremen [71].

$$\begin{gathered} \Delta K_{eff} = K_{app,max} - K_{op} \text{ if } K_{op} > K_{app,min} \\ \Delta K_{eff} = K_{app,max} - K_{app,min} \text{ if } K_{op} \leq K_{app,min} \end{gathered} \tag{3}$$

The crack opening stress intensity factor, K_{op}, accounts for both crack closure, according to [71], as well as the effects of residual stresses, as shown in Equation (4):

$$K_{op} = \min\left(\frac{0.2}{1 - R_{eff}}, 0.28\right) \cdot (K_{app,max} + K_{res}) \tag{4}$$

where R_{eff} is an effective stress ratio according to Equation (5).

$$R_{eff} = \frac{K_{app,min} + K_{res}}{K_{app,max} + K_{res}} \tag{5}$$

The weight function method was chosen for calculation of the stress intensity factors, with the general form as shown in Equation (6). Closed-form solutions for semi-elliptical surface cracks in finite plates presented in [72] and [73] were used for the weight functions, $m(x,a)$.

$$K = \int_0^a \sigma(x) \cdot m(x,a) \cdot dx \tag{6}$$

The chosen weight functions required the crack development at the surface, c, for which an empirical model of the crack shape, a/c, was chosen according to [74], which corresponded well with beach marks measured for both as-welded and HFMI-treated specimens, see Figure 16. The initial crack shape, a_0/c_0, was chosen based on comparisons to the experiments, on which more elaboration is made in Paper **III**.

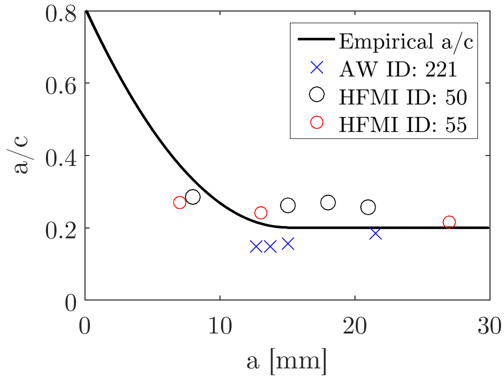

Figure 16. Crack shape development according to [74] compared to beach mark measurements on three specimens.

For calculation of the minimum and maximum applied stress intensities, the stress distributions, $\sigma(x)$, were computed for each series with the finite element method, including the actual geometries and toe radii. An example of the stress distribution for test series 1 is shown in Figure 17. The residual stresses were assumed to zero for the calculations of as-welded specimens, whereas for HFMI-treated specimens, compressive residual stresses were assumed near the surface as shown in Figure 18, based on the residual stress measurements.

To convert the fatigue strengths to axial loading, first, the number of cycles calculated for the propagation phase in bending were subtracted from the total number of cycles obtained from experiments. This can be considered as the crack initiation life. Subsequently, the number of cycles for crack propagation under axial loading was calculated and added to the initiation life to obtain the total fatigue life under axial loading. New *S-N* curves were constructed with the modified data and it was shown that the effect of loading mode on the fatigue strength is negligible for the specimen geometries studies in here. The modification procedure above assumes that the loading mode does not affect the crack initiation life of the specimens.

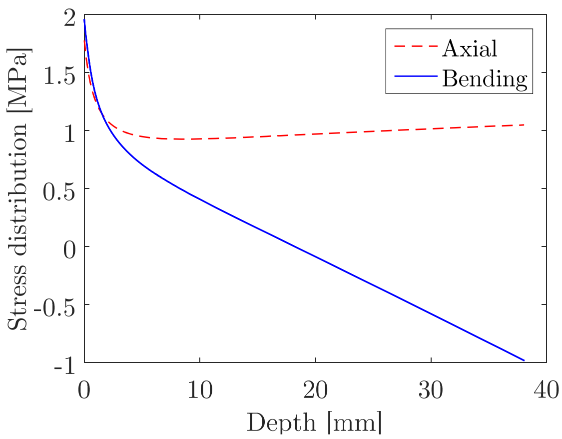

Figure 17. Stress distribution for 1 MPa nominal stress for series 1.

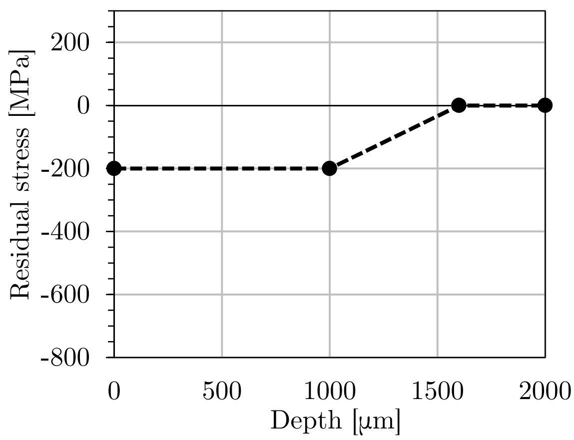

Figure 18. Assumed residual stress distribution for HFMI-treated specimens in LEFM calculations.

6 Conclusions

The aim of this thesis was to investigate the fatigue performance of high-frequency mechanical impact (HFMI) treated welded joints under conditions relevant for bridges. Studies were conducted on the potential for material saving by post-weld treatment of bridge welds and the thickness effect of HFMI-treated joints was investigated. The following main conclusions can be drawn based on the studies in this thesis:

- A material saving of around 20% is possible for the main load-carrying members of bridges. This could be achieved only with the application post-weld treatment, without using higher steel grades, on a railway bridge where the fatigue limit state dominated the design. In composite steel/concrete bridges, the post-weld treatment was calculated in conjunction with higher steel grades. In general, a steel grade of S460 was considered appropriate for bridge application after post-weld treatment since vertical deflection and instability phenomena can become limiting.

- The stress ratio is low ($R = 0.1$) for bridges where the self-weight is low compared to the traffic load, e.g. railway bridges. For heavy bridges in relation to the traffic load, stress ratios of up to $R = 0.7$ was calculated. In such cases, HFMI treatment can be effective if applied on site, after application of the stresses from self-weight.

- Longitudinal flange-to-web welds are common in bridges. These joints may not be post-weld treated and have a FAT-class of 125 at the best in the as-welded state. This limits the fatigue strength improvement that can be utilised in design.

- On the basis of the established database of HFMI-treated specimens, the thickness correction exponent for butt welds was calculated to $n = 0.055$. For non-load-carrying transverse attachment joints, $n = 0.207$ was obtained. Non-load-carrying longitudinal attachment joints showed a reverse thickness effect of $n =$ -0.188. These exponents are lower than what is given by the IIW for as-welded joints as well as for HFMI-treated joints in the cases of butt welds and longitudinal attachments.

- For HFMI-treated non-load-carrying transverse attachment joints with constant attachment thickness and weld size, $n = 0.135$ was derived on the basis of fatigue experiments conducted in this thesis. The fatigue test specimens comprised main plate thicknesses of 40 and 60 mm and showed significantly greater fatigue strength compared to recommended values by the IIW.

6.1 Suggestions for further research

Based on the aims of this thesis, the following subjects are proposed for further research:

- Characterization of realistic bridge loads by means of stress spectrums and identification of maximum and mean stresses in different bridge types for application in further fatigue experiments with variable amplitude loading.

- Application of HFMI treatment under different load levels, simulating the dead load in different bridge types, both experimentally and by finite element simulations.

- Fatigue experiments with crack initiation and growth monitoring to strengthen the notion of the improvement mechanisms of HFMI treatment and for verification of fatigue life prediction models for both the crack initiation and propagation phases.

- Establishment of a finite element modelling procedure to assess influential parameters on the fatigue life of HFMI-treated welds from a quality assurance point of view.

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