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FATIGUE LIFE IMPROVEMENT OF WELDED BRIDGE DETAILS USING HIGH FREQUENCY MECHANICAL IMPACT (HFMI) TREATMENT

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Abstract: Post weld treatment (PWT) techniques are used as measures to enhance the fatigue performance of steel and aluminum structures. These techniques have proven beneficial in various applications such as submarine hulls, offshore wind platforms and cranes. High Frequency Mechanical Impact (HFMI) treatment enhance the fatigue life of weldments by reducing the notch stresses, hardening the metal surface and inducing compressive surface residual stresses. This paper gives a short presentation of the HFMI technology and examples of their application in steel bridges. A feasibility assessment and a parametric study on the potential of material saving with PWT on steel bridges is also presented.

1 Introduction

During recent years, it has become harder for steel and composite bridges to compete with other materials mainly due to stricter requirements on the fatigue design, making it the dominant design factor for railway as well as road bridges. Fatigue governing the design has also caused limited use of modern high strength and ultra-high strength steels in bridge structures. Although experiences regarding PWT have not come far in the bridge industry, it is predicted that considerable economic advantages and positive effects on the bridges' life cycle costs can be achieved using PWT, also benefitting the environment. Case studies performed at Chalmers University of Technology indicate that large savings, with more than 20% material reduction, can be achieved for medium spanned railway bridges and highway composite bridges [1], by implementing PWT. Enhancing the fatigue strength of few critical details in the bridge also make it possible to use steels with higher strength than what is typically relevant in today's designs. In this paper, existing guidelines on fatigue improvement assessment for the HFMI technology are presented and examples of nine new and existing bridges are given where PWT has been applied. In addition, a feasibility assessment on the benefits of PWT is presented together with a parametric study of simply supported railway bridges.

Post weld treatment

There are several different categories of PWT techniques, some of the most common being burr grinding (BG), TIG dressing (TIG) and peening methods such as hammer and needle

peening. A more recent development in the field is the use of HFMI treatment. In general, all PWT methods enhance the fatigue strength through two main mechanisms:

1. Smother geometric weld/plate transition in the area of weld toe, where fatigue cracks are expected to initiate
2. Removal of surface weld defects (such as undercut) from the toe area.

Both mechanisms contribute to reduced stress concentrations and a positive shift of fatigue strength properties towards those of plain, non-welded details. The crack initiation phase becomes longer and leads to fatigue strength improvement. Due to a longer crack initiation phase, better steel qualities can be used, which additionally improves the fatigue strength of PW-treated details [2].

The peening methods, such as HFMI treatment, give additional advantages by introducing compressive residual stresses around the weld toe area. In as-welded details, the weld toe area usually experiences considerable tensile residual stresses, which are unfavorable in terms of fatigue. Thus, altering the residual stresses into favorable compressive stresses leads to higher fatigue strength. In a comparison between different PWT techniques, carried out by Pedersen et al [3], the HFMI treatment was shown to give the highest improvement for high cycle fatigue compared to burr grinding and TIG dressing. They additionally concluded that HFMI treatment was easy to apply and the fastest treatment.

2 High Frequency Mechanical Impact treatment

HFMI treatment belongs to the category of peening techniques and is similar to hammer and needle peening. However, HFMI provides greater quality due to higher frequency of the impacts, giving smaller spacing between indentations and resulting in better fatigue improvement. The manufacturers involved in HFMI technology label their tools using different names, such as UIT, HiFIT, PIT and UP etc. All of them are considered to give results within boundaries of the existing guidelines [2].

Improvement assessment guidelines

Marquis et al [2] presented a design proposal for HFMI-improved weld details, taking into account the benefits of increased steel quality. Decisive criteria that reduce the degree of improvement are identified as plate thickness and weld size effects as well as loading effects, such as stress ratio and variable amplitude loading. Plate thickness and weld size affect the stress concentration at the weld toe and must be taken into account when performing nominal stress and structural hot spot stress assessments. In variable amplitude loading, compressive overloads close to the materials yield stress can be harmful, as they may relax the beneficial residual stresses obtained by HFMI treatment. Therefore, such compressive overloads must be limited and the degree of improvement reduced for high stress ratios. Further, it is proposed that in a general case, HFMI treating details with steel qualities ≤ 355MPa can increase the fatigue strength with four fatigue (FAT) classes for the nominal stress assessment method, see Fig. 1a. When higher steel quality is used, an additional increase of one FAT class is suggested for every 200MPa of rise in material strength, see Fig. 1b. This can be compared to current recommendations by the International Institute of Welding (IIW) for hammer and needle peening which give an increase of two FAT classes for steel of ≤ 355MPa and tree FAT classes for > 355MPa [2]. Improvements by PWT can only be utilized on details with fatigue classes between 50 and 90 [2]. The reason is that other fatigue classes regard details which are

either non-welded, have a failure mode other than the weld toe, or where fatigue strength improvement are already accounted for in the fatigue class, e.g. ground flushed butt welds.

In Fig. 1, characteristic S-N curves suggested for HFMI-improved details are presented [2]. For comparison, an S-N curve for as-welded FAT 90 is also included. It is noted that the best improvements are gained for high numbers of cycles while for low numbers of cycles, no improvement is gained, which is indicated with the vertical dashed line.
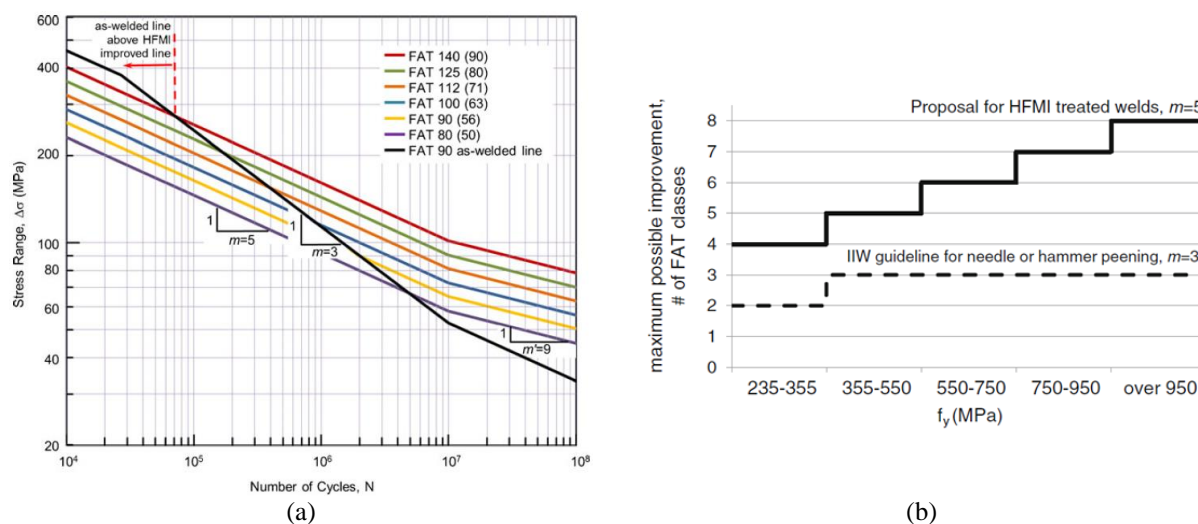

(a)   (b)

Fig. 1: (a) S-N-curves with an increase of four FAT classes for steel qualities $\leq 355\text{MPa}$. Values in parenthesis represent FAT class before treatment; (b) Stepwise increase for steel qualities [2].

3 HFMI applications on bridges

Three examples of application of HFMI on new bridges are presented here, see Table 1. Table 1 also lists six examples of HFMI application for repairing existing bridges. Brief presentations are given for the new bridges below. More details on these cases can be found in [4].

Table 1. Examples of bridges treated with HFMI [4].

#	Bridge	Location	Type	HFMI Technology	NEW	RE-PAIR	Ref.
1	Schenkendorfstrasse	Germany (Munich)	Tram	HiFIT	x		[5]
2	Bridge over A73 Autobahn	Germany (Thuringia)	Traffic	UIT	x		[6], [7], [8]
3	Cable car guide-way	USA (Las Vegas)	Cable car	PIT	x		[6],[9]
4	Ruhrstrom	Germany (Mülheim)	Railway	HiFIT		x	[10]
5	Bridge over Ohio River	USA (Louisville)	Highway	UP		x	[11],[12]
6	Bridge over Dnepr River	Ukraine	Railway	UP		x	[12]
7	Gschnitztal	Austria (Brenner)	Highway	PIT		x	[13],[14]
8	George Wade Memorial	USA (Pennsylvania)	Highway	UIT		x	[15]
9	Burignon	Switzerl. (Chardonne)	Railway	HiFIT		x	[16]

The Schenkendorfstrasse Bridge in Munich [5] was completed in 2009. The HiFIT treatment was considered early in the design stage of this bridge. The bridge has a cable-stayed superstructure with two interconnected decks, hanging from a pylon with three cable pairs, see Fig. 2. One of the decks is designated for tram traffic and the other for pedestrian traffic. The pylon itself is anchored partly to the southern end-support, and partly westward perpendicular to the bridge length. In order to meet the strict requirement of 100 years fatigue life for the heavy cyclic tram loads, this bridge relies on the fatigue strength enhancement achieved by HiFIT treatment. Only a limited number of fatigue-critical details in the superstructure, near its

northern end were treated. These details consist mainly of the welded flange–web connections of the main girders at the bridge end, connections between cross beams and main girders and connections to the supports. In Fig. 3, the construction phase is shown. As can be observed, treatment in the shop has the benefit of better control of work environment (lighting, operating position, improved control).

(a)   (b)

Fig. 2: Schenkendorfstrasse Bridge (Munich) from southern end (a) [17], and from east (b) [18].

(a)   (b)

Fig. 3: Schenkendorfstrasse Bridge during manufacturing [10]: (a) fabrication at workshop;
(b) Treatment of flange–web welds at workshop

Over the A73 Autobahn between Suhl–Lichtenfels in Germany, spans a 100m long tubular truss bridge, see Fig 4. This unique bridge is the first of its kind in Germany, since the nodes are formed by welding the tube profiles together, instead of, as usual for tubular bridges, being pre-cast as an individual unit. The bridge is made of steel grade S355 and was designed based on UIT fatigue enhancement of 32 of the highest stressed nodes [6]. More information on this bridge is found in [7] and [8].

Fig. 4: Bridge over A73 Autobahn between Suhl and Lichtendels (Germany) [7].

Based on previous positive experience of PIT treatment of tubular guide-ways in Oakland Airport's elevated tram tracks (CA), this fatigue enhancement technique was also used in the design for cable car guide-ways of tubular frame girders in Las Vegas, US [9], see Fig. 5. The fatigue requirements for such structures are stricter when over-bridging roads and motorways, according to American codes. Therefore, PIT treatment was used to meet these requirements without changing the original design. Laboratory tests conducted at University of Stuttgart and University of Seattle (WA) showed an increase of the fatigue life with a factor of 4.5 after PIT was implemented.

Fig. 5: Cable car guide-ways in Las Vegas (US) and conducted laboratory tests [6].

4 Feasibility assessment for bridges in Sweden

In this section, four different Swedish bridges are considered to assess the potential benefits of HFMI and steel quality on the amount of material and cost saving. One of the studied bridges is a steel railway bridge and the rest are highway composite steel and concrete bridges. The load effects and corresponding resistances of the bridges are evaluated and thereafter the bridges are re-designed by using higher steel grades or PWT, in order to optimize the use of material. The original bridges were designed with steel strength of 355MPa. The PWT is accounted for by assuming a fatigue strength enhancement of three FAT classes for the critical details in the bridge, according to the Eurocode part 3-1-9.

The first bridge in the study (1) is a simply supported railway (RW) bridge with a span of 24 m, over Östra Klarälven in Ställdalen-Kil. The second bridge (2) is a simply supported composite highway (HW) bridge with a span of 32 m, over E4 in Skulnäs. The third (3) is a three-span continuous highway bridge with constant cross section and spans of 13,8 – 11,3 – 13,3 m, over the River Nissan. The fourth bridge (4) is constructed as a tree-span continuous highway bride, with the same layout as bridge (3) but much larger spans and dimensions, in order to gain a holistic understanding of the potential benefits for a large variation of bridges. The spans of bridge (4) is 60 – 80 – 60 m and it has varying cross sections along the bridge.

Utilization ratios (URs) for different limit states are summarized for the original and new designs in Table 2 and Table 3. The limit states are abbreviated in the following manner: ULS = Ultimate limit state, SLS = Serviceability limit state and FLS = Fatigue limit state. Cost savings are given in the tables by subtracting the investment costs (cost of PWT and higher steel quality) from the material cost saving.

Table 2. Simply supported bridges, material and cost savings [1].

#	Type	URs – New design (Original design)				PWT	HSS	% steel saving	% cost saving
		ULS Bending	ULS Shear	SLS Deflection	FLS Damage				
1	RW	0,73 (0,53)	0,44 (0,36)	0,75 (0,54)	0,87 (1,03)	x		19	17
2	HW	0,90 (0,95)	0,74 (0,77)	0,95 (0,90)	0,85 (0,49)		x	17	7

Table 3. Continuous bridges, material and cost savings [1].

#	Type	Section	URs – New design (Original design)				PWT	HSS	% steel saving	% cost saving
			ULS Bending	ULS Shear	SLS Deflection	FLS Damage				
3	HW	Span	0,86 (0,76)	0,74 (0,72)	0,75 (0,51)	0,2 (0,18)	x	x	25	15
		Support	0,88 (0,87)	0,93 (0,91)	-					
4	HW	Span	0,82 (0,85)	0,88 (0,93)	0,87 (0,75)	0,4 (1,03)	x	x	23	14
		Support	0,82 (0,81)	0,83 (0,92)	-					

One can observe that for bridge (1) the material was poorly utilized in the original design since FLS dominated with a large margin compared to ULS and SLS. Since SLS dominated in the new design, the steel quality was not changed. On the contrary, for bridge (2) ULS dominated the design, closely followed by SLS. Thus, only increasing the steel strength was relevant for this case. Bridge (3) and (4) were dominated by ULS and FLS, respectively; nevertheless, a combination of PWT and increase of steel strength was shown to be feasible. If the recommendations given for HFMI had been used, the benefits of PWT would increase even more, since the slope of the S-N curves for HFMI-treated details are suggested to be flatter (m=5 instead of 3). More details on this study are given in [1].

5 Parametric study of railway bridges

A more general parametric study on the potential benefit of PWT in the design of simply-supported steel railway bridges has also been performed. The investigation is based on single-track railway bridges with spans varying between 10 and 30 meters. The utilization ratio in design limit states are compared and the potential for material savings are evaluated.

As a starting point, preliminary bridge cross-sections are produced with the moment of inertia equal to what is required by a deflection restriction of L/800 in SLS. This procedure is done in order to ensure that the original designs will fulfill the deflection criterion, since this is the limit which cannot be affected, other than changing boundary conditions. The main parameter that varies for different spans is the web height, although the web thickness and the width of the bottom flanges are also allowed to vary to some extent; all other dimensions are kept constant. A typical cross-section is visualized in Fig. 6.
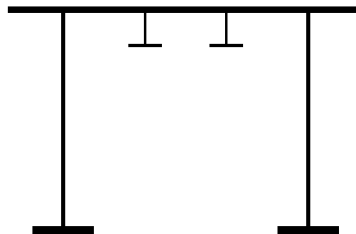

Fig. 6: Typical cross section used in the parametric study of simply-supported railway bridges.

Thereafter, calculations are performed for the ULS and FLS. In the ULS, the bridge is verified with regard to bending, shear, combination of bending and shear, and web breathing for the train load model 71 according to Eurocode part 1-2. The FLS is calculated by the linear damage accumulation method for an attachment of a "non-load carrying" vertical stiffener welded to the flanges in the mid-span. Heavy traffic mix of trains (Eurocode part 1-2) is used and fatigue life is set to 100 years. The safety factor γ_{Mf} is equal to 1.35. It is observed that FLS becomes greatly dominating, see Fig. 7.
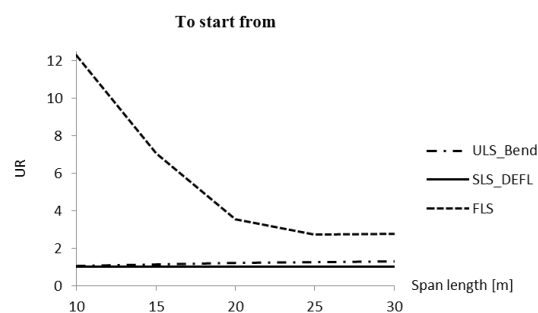

Fig. 7: Utilization ratios for the preliminary cross-sections.

Subsequently, flange thicknesses and web heights are increased for all spans so that utilization factors for all the limit states fall below unity. This results in the dimensions which are used as the original designs, with the top and bottom flange thicknesses equal to 35mm and 45mm, respectively, in all cross-sections. The utilization ratios for the original designs are presented in Fig. 8a.

In the original designs, Fig. 8a, the FLS govern the design and a UR around 100% is obtained for all bridge spans. As expected, fatigue is more governing for shorter spans and thus the other limit states (i.e. ULS and SLS) are well below unity for shorter spans, but become more prominent when the span length increases. Employing a three-class enhancement of the critical detail brings down the UR for FLS to around 30%, see Fig. 8b. Consequently, for bridge spans which are not governed by SLS-requirements, the steel strength can be increased and a reduction in section modulus can be obtained through a new design, i.e. material saving. The material strengths is increased from 355MPa to 460MPa, in Fig. 8c.

A new design is made, optimized with regard to the governing limit state after the use of steel 460MPa and PWT, see Fig 6 d). The new design is made by reducing flange thicknesses to 20mm and 30mm for the top and bottom flanges, respectively, and minor changes of the width of the lower flanges.
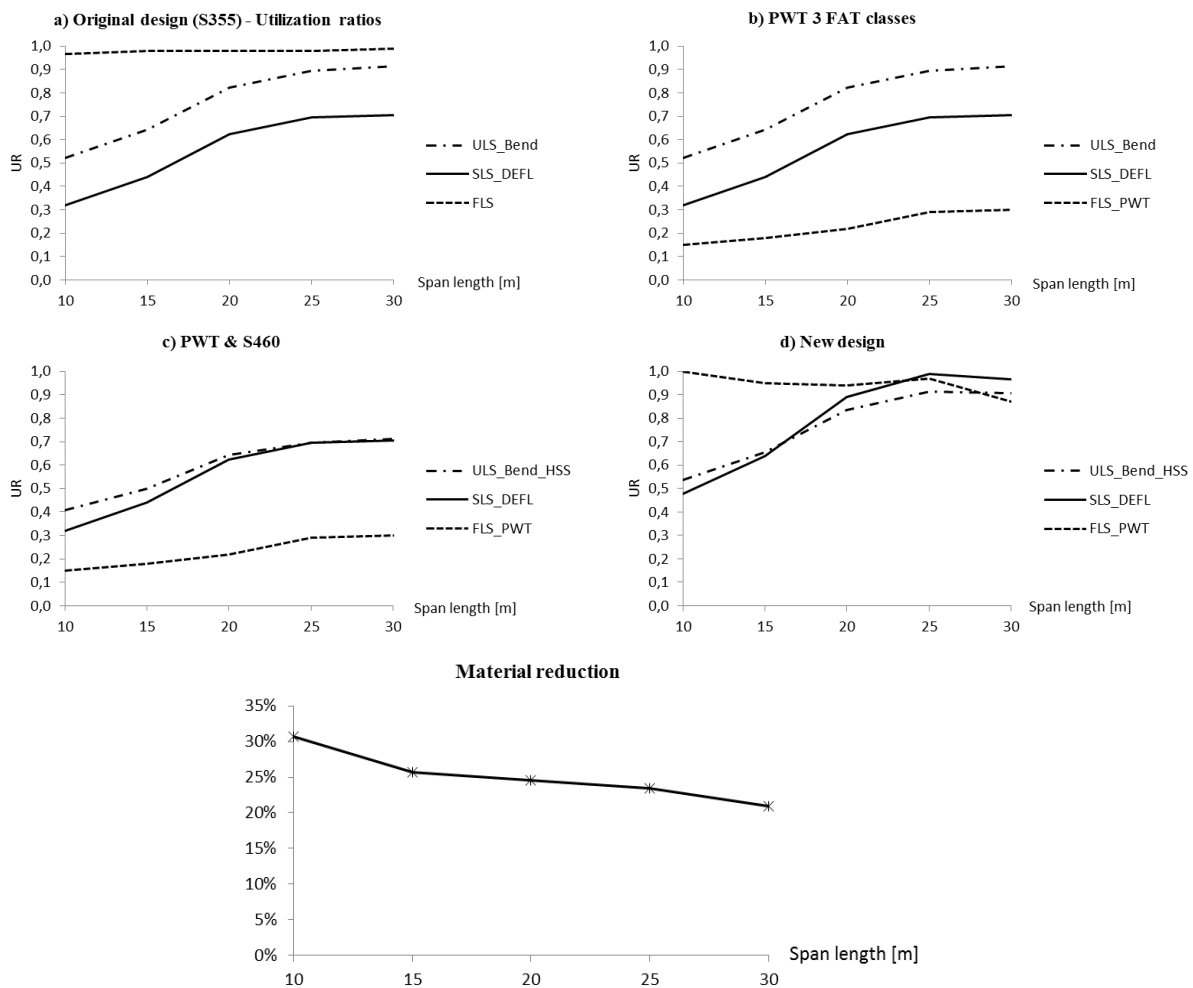

Fig. 8: Parameter study with utilization ratios for ULS, SLS and FLS, of simply supported railway bridges with spans between 10 and 30 meters.

From this parametric study, it is seen that there is an obvious potential for reducing the material use in a bridge by using PWT. The material reduction obtained is higher for shorter span lengths (fatigue is more governing) but even for longer spans, a substantial saving can be obtained. A more accurate investigation of real bridges with different span lengths would however give more accurate results of the possible material savings.

6 Conclusions

HFMI can significantly improve the fatigue strength of welded structures, in particular when steel material with yield strength higher than 355MPa is used. This method gives the best results for high cycle fatigue due to the S-N curves with lower slopes (m=5). However, the risk of relaxing the induced benefitial residual stresses must be considered when there are potential of compressive overloads.

In this paper, examples of nine bridges are given where HFMI was employed, both in the design of new bridges, and as a measure to repair and refurbish.

Feasibility assessments and a parametric study on the potential benifits of applying HFMI-treatement to enhance the fatigue strength of welded details in steel bridges have been presented. The results show that a substantial saving in material can be obtained by treating few critical details in the bridge. In addition, the PWT allow for the use of steel with higher strength than what is conventionaly used in bridges today. This gives the possibility for additional weigth and material reduction.

The benifits of PWT can be realized in the design of both road and railway bridges. For railway bridges, which are considered in the parametric study, the material reduction varies between 30% for short-span bridges and 20% for bridges with spans of 30m, when the fatigue strength is increased with three classes. No account is taken in this study for the change in slope of the S-N curves of PW-treated details, which should give additional saving. In addition, if a fatigue strength improvment of more than three classes can be realized, further reduction in material can be obtained for bridges with spans less than 25m.

Acknowledgments

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