High Frequency Mechanical Impact Treatment
- Recommendations for the design of welded details in road and railway bridges

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1. Introduction

1.1 Background

High Frequency Mechanical Impact treatment (HFMI) is a post-weld treatment method that can be used to enhance the fatigue strength of welded details. The term HFMI covers several different high frequency peening techniques and equipment's, which come in different commercial names, such as ultrasonic impact treatment (UIT), ultrasonic peening (UP), high-frequency impact treatment (HiFIT), etc. Common for all these techniques is that indenters of hardened high strength steel are used to impact and deform the steel material at the weld toe region with high frequency. This results in a considerable increase of fatigue resistance with respect to fatigue cracking from weld toe. The enhanced fatigue strength is generally obtained through three main mechanisms:

1. *Geometry*: smother transition in the weld toe area resulting in reduced stress concentration.
2. *Material*: increase in material hardness due to the cold working, and thus increased resistance to crack initiation.
3. *Residual stress*: more importantly, the production of local compressive residual stress field at and around the weld toe

These mechanisms produce a positive shift of fatigue strength properties towards those of plain, non-welded details.

For the same reasons, the fatigue performance of HFMI-treated details differs from that of their as-welded counterparts in several ways:

- The fatigue strength of HFMI-treated welded details becomes <u>dependent on the steel yield strength</u>. Details made of higher strength steel will exhibit higher fatigue strength after HFMI-treatment
- The fatigue performance of these details also becomes <u>dependent on the stress ratio</u> (or mean stress). Load cycles with higher R-ratios (or mean stress) become more damaging than cycles with the same stress range but with lower mean stress
- Unlike welded details in as-welded condition, HFMI-treated welds are <u>sensitive to overloads</u>. Particularly load cycles with compressive peeks can be determinantal as they may result in relaxation of the beneficial compressive residual stresses generated by the HFMI process.

Therefore, fatigue design and analysis of HFMI-treated welds should consider all three forementioned effects and the fatigue verification procedure as outlined in the Eurocodes for details in as-welded condition should be modified accordingly.

This guideline document is the result of the research conducted on the topic at Chalmers University of Technology during the years 2015-2023. In addition to the derivation of fatigue resistance properties of HFMI-treated details, this work has resulted in a complete design methodology that can be used in the design of road and railway bridges with HFMI-treated details. The document also includes general requirements on welds before HFMI-treatment as well as recommendations for checks and quality assurance of the treatment. More detailed background information can be found in various scientific publications that are listed in Section 7.

1.2 Scope and limitations

The bridge industry has shown a great interest in utilizing the benefits of HFMI-treatment both in the design of new bridges and for the purpose of fatigue life extension of existing ones. The aim of this document is to facilitate and support a safe application of HFMI treatment on steel and composite road and railway bridges. The design method outlined in this guidelines document is compatible with the rules stipulated in Annex F of the upcoming (updated) version of EN 1993-1-9. As such, the following limitations are adopted:

[1] Only construction details stipulated in Table 1 are covered by this document. For these details, the fatigue strength values refer to toe cracking. Root cracking does not need to be checked.

[2] The fatigue strengths assigned to different details in this document are only applicable to welded constructional details with plate thicknesses $t \geq 5$ mm.

[4] This document applies to steels covered by EN 1993-1-1 with yield strengths S235-S700 as well as equivalent older steels that fulfil the ductility requirements for modern steels with the same strength. It does not apply to weathering steels according to EN 10025-5 or stainless steels according to EN 10088. Verification of the performance of HFMI on welded details in these two materials is ongoing.

[5] Fatigue strength values stipulated in this document are valid for qualified post-weld treatment technologies that have demonstrated to give comparable results. In accordance with Annex F of the upcoming updated version of EN 1993-1-9, qualified post-weld treatment technologies (tools) whose effectiveness has been proved by experimental research are:

HiFiT (High Frequency Impact Treatment),

PIT (Pneumatic Impact Treatment),

UIT (Ultrasonic Impact Treatment).

Other HFMI-technologies can be used if similar fatigue performance can be demonstrated by testing.

The recommendations given in this document covers the application of HFMI-treatment in:

- Design of new road and railway bridges
- Strengthening of new bridges with new welded details
- Fatigue life extension of existing bridges by means of HFMI-treatment of existing welds

Additional limitations and requirement for the applicability of the models proposed in this guidelines document may be given when relevant through the text of this document.

1.3 References to relevant standards

The design models given in this document for road and railway bridges are derived to be used along with the different load models currently stipulated in *EN 1991: Actions on structures*. These design models are also compatible with the fatigue design models using the simplified λ-coefficients method or the cumulative damage method according to *EN 1993-2, Design of Steel Structures — Part 2: Steel bridges*. The fatigue verification format follows the one given in *EN 1993-1-9, Design of Steel Structures — Part 1-9: Fatigue strength of steel structures*, with modifications made when necessary to account for the effect of yield stress and mean stress in the design of HFMI-treated details. Finally, reference is made to *EN 1090-2* and *EN ISO 5817* concerning requirements on weld quality before HFMI-treatment.

The IIW-recommendation on HFMI-treatment [9] can also be consulted for more detailed view of the HFMI method and process as well as quality control and safety.

1.4 Symbols

CAFL	Constant Amplitude Fatigue Limit
D	accumulated fatigue damage due to different stress ranges ($D = \sum Di$)
D_i	fatigue damage due to a stress range $\Delta\sigma_{i,E}$ with $N_{i,E}$ stress cycles ($D_i = N_{i,E} / N_{i,R}$)
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
m_1, m_2	slope parameter of a fatigue resistance curve
N_{eq}	equivalent number of cycles from variable amplitude loading
R	the ratio of minimum to maximum stress in a stress cycle
$\Delta\sigma_C$	characteristic fatigue resistance in as-welded condition at 2×10^6 cycles
$\Delta\sigma_{C,HFMI,ref}$	reference value for fatigue resistance of HFMI-treated detail (f_y = 355 MPa, R = 0,1)
$\Delta\sigma_{D,HFMI.ref}$	reference value for CAFL (knee point) of HFMI-treated detail (f_y = 355 MPa, R = 0,1)
$\Delta\sigma_{C,HFMI}$	characteristic fatigue resistance for HFMI-treated detail at 2×10^6 cycles
$\Delta\sigma_{D,HFMI}$	characteristic constant amplitude fatigue limit (knee point) for HFMI-treated detail
$\Delta\sigma_{Ed}$	fatigue action design effect (stress ranges)
$\Delta\sigma_{e,2,HFMI,Ed}$	equivalent design stress range for HFMI treated joints, i.e. accounting for λ_{HFMI}
$\Delta\sigma_{eq}$	equivalent stress range for a variable amplitude loading
$\Delta\sigma_{eq,R}$	equivalent stress range for a variable amplitude loading accounting for R-ratios
$\Delta\sigma_p$	stress range from FLM3
σ_{perm}	stress from permanent loads
σ_{max}	maximum stress from the characteristic load combination
$\Delta\sigma_s$	characteristic stress range at intersection of fatigue resistance curves in as-welded and HFMI-treated state
γ_{Mf}	partial factor for fatigue resistance
γ_{Ff}	partial factor for applied stress ranges $\Delta\sigma_E$
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 load to stress range of the load model

2. Fatigue resistance of HFMI-treated welded details

2.1 Fatigue resistance curves

The characteristic fatigue resistance curve for a welded detail in as-welded and HFMI-treated condition are depicted in Figure 1 HFMI-treated details are assigned double-slope curves with slopes 5 and 9 with the knee point at 5×10^6 cycles.

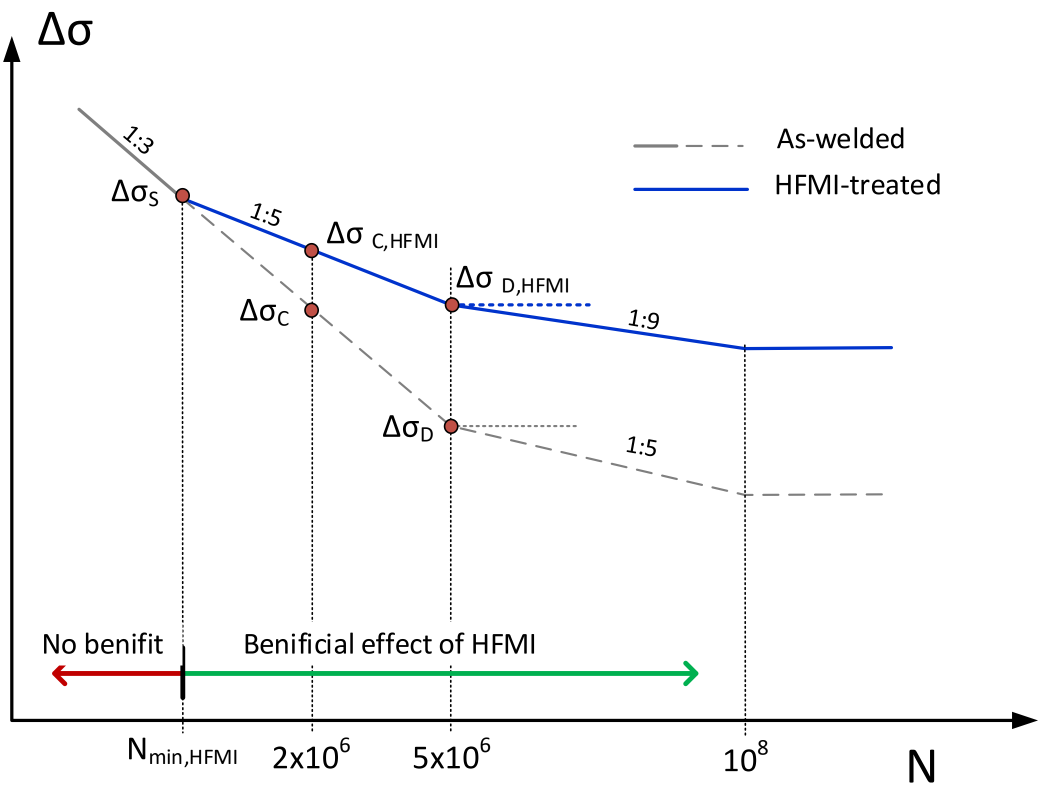


Figure 1: Representation of the S-N curves for a welded detail in as-welded and HFMI-treated conditions

$\Delta\sigma_{C,HFMI}$ refers to the fatigue strength of the detail after modification with reference to yield strength and – when applicable – R-ratio and thickness effect, see Section 2.3.

$\Delta\sigma_{D,HFMI}$ refers to the stress range at the knee point where the slope of the S-N curve for HFMI-treated detail changes from 5 to 9. It also designates the Constant Amplitude Fatigue Limit for HFMI-treated details. $\Delta\sigma_{D,HFMI}$ can be calculated from:

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

The cut-off limit is defined at 100 $\times 10^6$ cycles and is obtained from:

$$\Delta\sigma_{L,HFMI} = \left(\frac{5}{100}\right)^{\frac{1}{9}} \times \Delta\sigma_{D,HFMI} = 0.717 \times \Delta\sigma_{D,HFMI} \quad (2)$$

$\Delta\sigma_s$ and the corresponding number of cycles $N_{min,HFMI}$ refer to the stress range and, the number of cycles that limit the benefit of HFMI. The fatigue strength curve of an HFMI-treated detail can only be used if the applied stress range is below ${}^{\Delta\sigma_s}/{\gamma_{Mf}}$ or when the applied number of cycles is below $N_{min,HFMI}$., Otherwise, the S-N curve for as-welded detail should be used.

$N_{min,HFMI}$ and $\Delta\sigma_s$ are to be calculated using the following formulae:

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

$$N_{min,HFMI} = 2.10^6 \left(\frac{\Delta\sigma_C}{\Delta\sigma_s}\right)^3 = 2.10^6 \left(\frac{\Delta\sigma_{C,HFMI}}{\Delta\sigma_s}\right)^5 \tag{4}$$

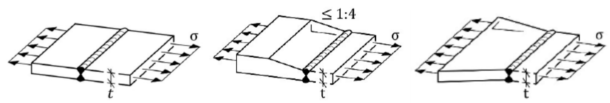
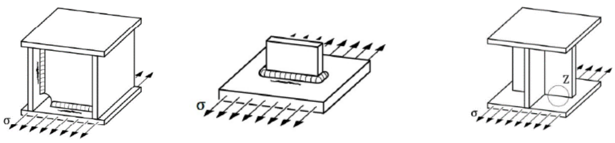
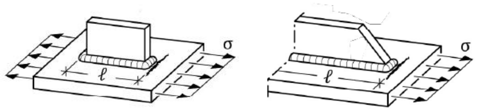
Where $\Delta\sigma_C$ is the detail category for the detain in the as-welded condition.

2.2 Classification of constructional details

The welded details that are covered by this document are listed in Table 1 along with the reference value of their fatigue strength, $\Delta\sigma_{C,HFMI,ref}$.

$\Delta\sigma_{C,HFMI,ref}$ denotes the *reference* value of fatigue strength of the constructional detail, which were derived from constant amplitude fatigue tests on welded details having steel with yield strength of 355 MPa performed at an R-ratio of 0.1. For fatigue verification, this reference value needs to be adjusted with reference to steel yield strength (f_y) and the stress ratio (R), see Section 2.3 and Chapter 3.

Table 1: Constructional welded details covered by this document

Detail	Description	$\Delta\sigma_{C,HFMI,ref}$
	Transverse K- and X-butt welds [1, 2]	$k_S \times 160$
	Transverse non-load carrying attachments and stiffeners, with fillet or butt welds.	140
	End of longitudinal non-load carrying attachments with fillet or butt welds	100

1) A thickness correction factor $k_S = \left(\frac{25}{t}\right)^{0.2}$ is applied when plate thickness exceeds 25 mm.
2) Plates of equal dimensions or with tapering in width or thickness with slop ≤ 1:4

***Note**: For high-strength steels, the fatigue strength can – after modification with the steel yield strength – become higher that the fatigue strength of the base metal. In such cases, verification of the fatigue strength of the base metal should be performed.*

2.3 Modification of fatigue resistance

The fatigue strength of HFMI-treated details is dependent on the steel yield strength (f_y) and the R-ratio (or mean stress). Before being used in the fatigue verification, the reference value of the fatigue strength (Table 1) needs to be modified with respect to f_y and *R-ratio*. The general format of modification is given in equation (5).

$$\Delta\sigma_{\mathrm{C,HFMI}} = f_1 f_2\, \Delta\sigma_{\mathrm{C,HFMI,ref}} \qquad (5)$$

Where f_1 is the modification factor accounting for the effect of yield strength f_y determined as follows:

$$f_1 = 1 + \frac{0.1(f_y - 355)}{\Delta\sigma_{\mathrm{C,HFMI,ref}}} \qquad (6)$$

and f_2 is the modification factor accounting for the effect of stress ratio R determined as follows:

$$f_2 = \frac{1}{0.5R^2 + 0.95R + 0,9} \quad \text{if } 0.1 < \mathrm{R} < 1.0\text{, otherwise, } f_2 = 1.0 \qquad (7)$$

***Note**: When calculating the modification factor f_1, the nominal value for yield strength f_y may be used without correction for plate thickness.*

***Note**: For road and railway bridges, account is taken for the effect of varying R-ration from traffic loading via the correction factor λ_{HFMI}, see sections 3.2.*

3. Fatigue verification

3.1 Constant amplitude fatigue loading

For the special case of details subjected to constant amplitude loading, fatigue verification can be made by relating the design stress range to the fatigue strength value of the detail. The verification format becomes:

$$\frac{\Delta\sigma_{\mathrm{Ed}}}{\Delta\sigma_{\mathrm{C,HFMI}}/\gamma_{Mf}} \leq 1.0, \text{ and } \Delta\sigma_{\mathrm{Ed}} < \frac{\Delta\sigma_{s}}{\gamma_{Mf}} \quad (8)$$

With

$$\Delta\sigma_{\mathrm{Ed}} = \Delta\sigma_{\mathrm{E}}\gamma_{Ff} \quad (9)$$

Where $\sigma_{\mathrm{C,HFMI}}$ is the modified fatigue strength according to equation (5) and $\Delta\sigma_{\mathrm{E}}$ is the constant amplitude fatigue action effect.

3.2 Variable amplitude fatigue loading in road and railway bridges

HFMI-treated details in road and railway bridges are subjected to variable amplitude loading. The effect of the stress ratios (generated by traffic loads and permanent stresses) should be accounted for by magnifying the design stress range obtained from traffic load models with the factor λ_{HFMI}, see section 3.2.3.

The verification format depends on the method adopted for fatigue design and the corresponding load models.

3.2.1 Fatigue verification using the λ-coefficients method

When fatigue design of welded HFMI-treated details is performed using the simplified λ-coefficients method with FLM3 and LM71 for road and railway bridges, the verification format is:

$$\frac{\Delta\sigma_{e,2,HFMI,Ed}}{f_1 \times \Delta\sigma_{C,HFMI,ref}/\gamma_{Mf}} < 1 \quad (10)$$

$\Delta\sigma_{\mathrm{C,HFMI,ref}}$ is obtained from Table 1 and f_1 is the factor taking the effect of the material yield strength into account, see equation (6).

$$\Delta\sigma_{\mathrm{e,2,HFMI,Ed}} = \lambda_1\lambda_2\lambda_3\lambda_4\lambda_{HFMI}\,\Delta\sigma_{\mathrm{Ed}} \quad (11)$$

but

$$\lambda_1\lambda_2\lambda_3\lambda_4 < \lambda_{Max}$$

The damage equivalent factors λ_1 to λ_4 and λ_{max} are as specified in EN 1993-2.

λ_{HFMI} is a damage equivalent factor that considers the effect of stress ratio from permanent load and real traffic load on bridges. As such, the reference fatigue strength $\Delta\sigma_{\mathrm{C,HFMI,ref}}$ does not need to be corrected with the factor f_2.

For calculation of λ_{HFMI} see Section 3.2.3.

***Note**: the factors λ_2 and λ_4 are to be calculated using an exponent of 5 for HFMI-treated details in accordance with EN 1993-2 .*

3.2.2 Fatigue verification using the method of damage accumulation

When a set of vehicles (FLM4) or a set of trains (train mix) is used in fatigue verification using the damage accumulation method, the verification of HFMI-treated details reads:

$$D = \frac{\Sigma n}{N_{eq}} \leq 1.0 \tag{12}$$

$\Delta\sigma_{eq}$ which corresponds to N_{eq}, is the equivalent stress range generated by the load model containing several vehicles, i.e. FLM4 for road bridges and train mix for railway bridges according to EN 1991.

$$\Delta\sigma_{eq} = \begin{cases} \sqrt[m_1]{\dfrac{\Sigma(n_i\,\Delta\sigma_i^{m_1}) + \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}\right)^{m_1-m_2} . \Sigma(n_j . \Delta\sigma_j^{m_2})}{\Sigma n_i + \Sigma n_j}}, \text{ if } \dfrac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} \leq \Delta\sigma_{eq} \\ \sqrt[m_2]{\dfrac{\Sigma(n_i\,\Delta\sigma_i^{m_1}) . \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}\right)^{m_2-m_1} + \Sigma(n_j . \Delta\sigma_j^{m_2})}{\Sigma n_i + \Sigma n_j}}, \text{ if } \dfrac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} > \Delta\sigma_{eq} \end{cases} \tag{13}$$

and the equivalent number of cycles is:

$$N_{eq} = \begin{cases} 5 \cdot 10^6 \left(\dfrac{\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}}{\lambda_{HFMI} \cdot \Delta\sigma_{eq} \cdot \gamma_{Ff}}\right)^{m_1}, \text{ if } \dfrac{f_1 \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} \leq \lambda_{HFMI} . \Delta\sigma_{eq} \\ 5 \cdot 10^6 \left(\dfrac{\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}}{\lambda_{HFMI} \cdot \Delta\sigma_{eq} \cdot \gamma_{Ff}}\right)^{m_2}, \text{ if } \dfrac{f_1 \cdot \Delta\sigma_{D,HFMI.ref}}{\gamma_{Mf}} > \lambda_{HFMI} . \Delta\sigma_{eq} \end{cases} \tag{14}$$

In equations (13) and (14)

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, $f_1 \Delta\sigma_{D,HFMI,ref} / \gamma_{Mf}$

In the special case, where the fatigue load models (FLM4 or train mixes) generate only (or predominantly) loading cycles with stress ranges that fall only above or below the knee point stress ($f_1 \Delta\sigma_{D,HFMI,ref} / \gamma_{Mf}$), the relevant expression for $\Delta\sigma_{eq}$ in equation (13) can be used directly so it is simplified to:

$$\Delta\sigma_{eq} = \sqrt[m]{\frac{\Sigma(n . \Delta\sigma_i^m)}{\Sigma n}} \tag{15}$$

And the equivalent number of cycles becomes:

$$N_{eq} = 5 \cdot 10^6 \left(\frac{f_1 \cdot \Delta\sigma_{D,HFMI,ref} / \gamma_{Mf}}{\lambda_{HFMI} \cdot \Delta\sigma_{eq} \cdot \gamma_{Ff}}\right)^m \tag{16}$$

Where m is the relevant slope (5 or 9).

3.2.3 Damage equivalent factor for R-ratio, λ_{HFMI}

To account for the variable R-ratio coming from real traffic on road and railway bridges, a damage equivalent factor λ_{HFMI} was derived from measured traffic data. The background and derivation of this factor can be found in [2-4].

Similar to the procedure used in EN 1993-2, λ_{HFMI} is calculated dependent on the location of the construction detail in the bridge, i.e. midspan or over intermediate support.

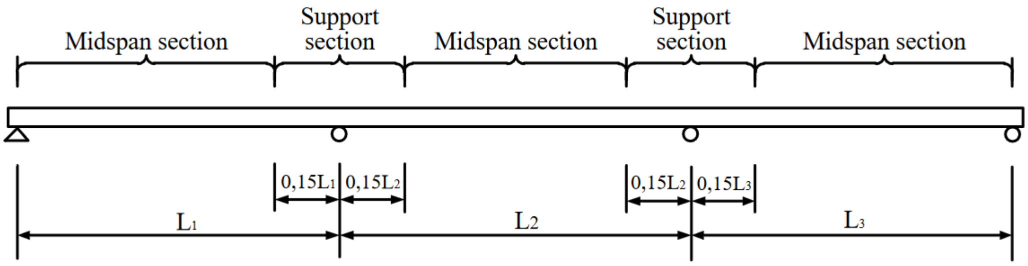


Figure 2: Definition of midspan and mid-support sections

For **road bridges** λ_{HFMI} is calculated according to:

$$\lambda_{HFMI} = \frac{2.38\Phi+0.64}{\Phi+0.66} \geq 1.0 \qquad \textit{Midspan} \tag{17}$$

$$\lambda_{HFMI} = \frac{2.38\Phi+0.06}{\Phi+0.40} \geq 1.0 \qquad \textit{Mid-support} \tag{18}$$

Φ is a factor that takes account for the acting permanent load effect on the R-ratio

$$\Phi = \frac{\sigma_{perm}}{2\times\Delta\sigma_p} \tag{19}$$

$\Delta\sigma_p$ is the stress range generated by the passage of FLM3.

σ_{perm} is the stress from permanent loads.

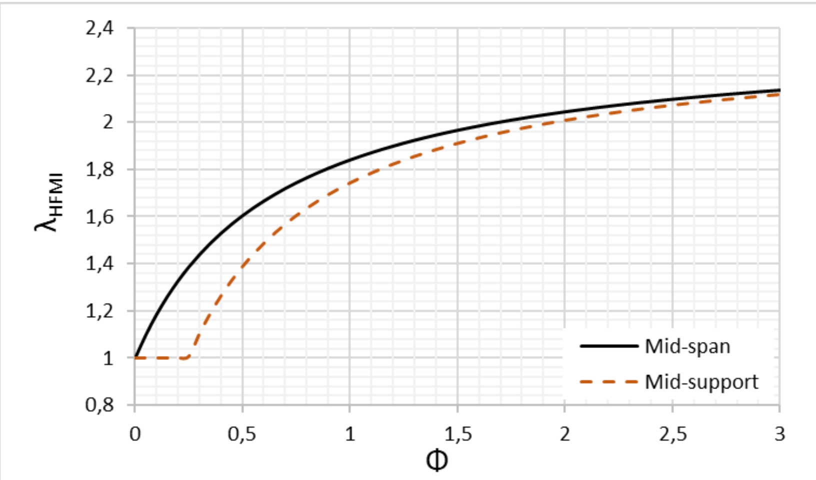


Figure 3: The damage equivalent section (λ_{HFMI}) for road traffic

Note: FLM3 is only used in order to calculate Φ, thus, the method itself is not limited to be used with this load model.

For **railway bridges** λ_{HFMI} is calculated according to:

$$\lambda_{HFMI} = \frac{2.38\Phi+1.18}{\Phi+1.07} \geq 1.0 \qquad \textit{Midspan} \tag{20}$$

$$\lambda_{HFMI} = \frac{2.56\Phi+1.12}{\Phi+1.61} \geq 1.0 \qquad \textit{Mid-support} \tag{21}$$

Φ: is a factor that takes account for the acting permanent load effect on the R-ratio

$$\Phi = \frac{\sigma_{perm}}{0.73\times\Delta\sigma_{LM71}} \tag{22}$$

when fatigue load model FLM71 is used in the verification, or

$$\Phi = \frac{\sigma_{perm}}{0.90\times\Delta\sigma_{max}} \tag{23}$$

when a train mix is used in the verification

$\Delta\sigma_{LM71}$ is the stress range generated by the passage of load model LM71.

$\Delta\sigma_{max}$ is the maximum stress range generated by any of the train models in the train mix (usually generated by train type 5 which exists in all Eurocode's train mixes).

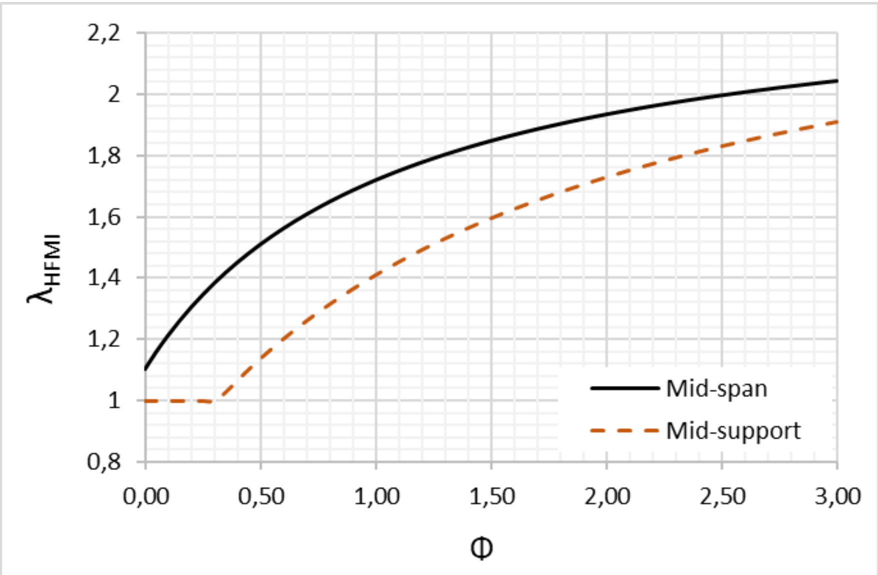


Figure 4: The damage equivalent section (λ_{HFMI}) for railway traffic

Note: If HFMI-treatment is applied after bridges erection, λ_{HFMI} should be calculated using Φ=0.

3.2.4 General approach for treatment of the R-ratio effect

In cases where bridges (or other types of fatigue loaded structures) are designed for other loads than those covered by fatigue load models in EN1991. A typical example is railway bridges on the *Malmbana* railway line in Sweden which is loaded by freight trains consisting of Fanoo -type wagons, to be designed using load model 13S.

In such situation, the effect of mean stress generated from train load imposed on stresses from permanent loads can be accounted for directly in the calculation of the equivalent stress. This is done by magnifying each stress range ($\Delta\sigma_i$) in the stress spectrum with its corresponding f_2 factor calculated according to equation (7), with R_i being calculated as:

$$R_i = \frac{\sigma_{min}}{\sigma_{max}} = \frac{\sigma_{min,i} + \sigma_{perm}}{\sigma_{max,i} + \sigma_{perm}} \tag{24}$$

σ_{perm} is the (static) stress from permanent load in the control section

$\sigma_{min,i}$ and $\sigma_{max,i}$ are the minimum and maximum stress in i^{th} stress cycle in the spectrum generated by the load model

With every stress range in the spectrum $\Delta\sigma_i$ and the corresponding R_i and $f_{2,i}$ calculated, the summation of total damage can then be calculated using $\Delta\sigma_{eq,R}$

$$\Delta\sigma_{eq,R} = \begin{cases} \sqrt[m_1]{\dfrac{\Sigma\left(n_i\left[\frac{\Delta\sigma_i}{f_{2,i}}\right]^{m1}\right)+\left(\frac{f_1.\Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}\right)^{m_1-m_2}.\Sigma\left(n_j.\left[\frac{\Delta\sigma_j}{f_{2,j}}\right]^{m2}\right)}{\Sigma n_i+\Sigma n_j}} & \text{if } \dfrac{f_1.\Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} \leq \Delta\sigma_{eq,R} \\ \sqrt[m_2]{\dfrac{\Sigma\left(n_i\left[\frac{\Delta\sigma_i}{f_{2,i}}\right]^{m1}\right).\left(\frac{f_1.\Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}}\right)^{m_2-m_1}+\Sigma\left(n_j.\left[\frac{\Delta\sigma_j}{f_{2,j}}\right]^{m2}\right)}{\Sigma n_i+\Sigma n_j}}, & \text{if } \dfrac{f_1.\Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} > \Delta\sigma_{eq,R} \end{cases} \tag{25}$$

Here $\Delta\sigma_{eq,R}$ is the equivalent stress range for a variable amplitude loading where each stress cycle $\Delta\sigma_i$ is corrected with its correction factor for R-ratio effect, $f_{2,i}$.

The equivalent number of cycles becomes:

$$N_{eq} = \begin{cases} 5\cdot10^6\left(\dfrac{\frac{f_1.\Delta\sigma_{D,HFMI.ref}}{\gamma_{Mf}}}{\Delta\sigma_{eq,R}.\gamma_{Ff}}\right)^{m_1} & if \quad \dfrac{f_1\Delta\sigma_{D,HFMI,ref}}{\gamma_{Mf}} \leq \Delta\sigma_{eq,R} \\ 5\cdot10^6\left(\dfrac{\frac{f_1.\Delta\sigma_{D,HFMI.ref}}{\gamma_{Mf}}}{\Delta\sigma_{eq,R}.\gamma_{Ff}}\right)^{m_2} & if \quad \dfrac{f_1.\Delta\sigma_{D,HFMI.ref}}{\gamma_{Mf}} > \Delta\sigma_{eq,R} \end{cases} \tag{26}$$

and the fatigue verification reads:

$$D = \frac{\sum n}{N_{eq}} \leq 1.0$$

***Note**: The same procedure above can be used in any situation where the stress cycles in the spectrum with their corresponding R-ratios are known, e.g. when the fatigue verification is made on the bases of measured stresses or detailed measured traffic loads.*

3.2.5 Derivation of λ_{HFMI}

The damage equivalent factors λ_{HFMI} given in this document for road and railway traffic were derived from analysis of measured traffic data from Sweden and the Netherlands. Similar factors can be derived for other fatigue loaded structures where the variation of R-ratio during operation of the structure is judged to be considerable.

Assuming that sufficient empirical operating loads (or load effects) are available, λ_{HFMI} can be calculated as:

$$\lambda_{HFMI} = \frac{\Delta\sigma_{eq,R}}{\Delta\sigma_{eq}} \tag{27}$$

where

$$\Delta\sigma_{eq} = \left(\frac{\Sigma(n_i \times \Delta\sigma_i{}^m)}{\Sigma n_i}\right)^{\frac{1}{m}} \tag{28}$$

and

$$\Delta\sigma_{eq,R} = \left(\frac{\Sigma(n_i \times (\Delta\sigma_i . f_{2,i})^m)}{\Sigma n_i}\right)^{\frac{1}{m}} \tag{29}$$

with m = 5 and

$$f_{2,i} = 0.5R_i^2 + 0.95R + 0.9, \qquad f_{2,i} \geq 1 \tag{30}$$

For more detailed information of the derivation of λ_{HFMI} refer to [2-4].

3.2.6 Treatment of multiaxial loading

For the general case in which a welded detail is subjected to a multiaxial loading situation, the fatigue verification should consider the summation of fatigue damage from the individual stress components (i.e. normal and shear stresses). In doing so, the enhancement of fatigue strength due to HFMI-treatment can only be accounted for with respect to the stress component perpendicular to the weld.

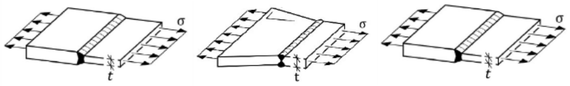
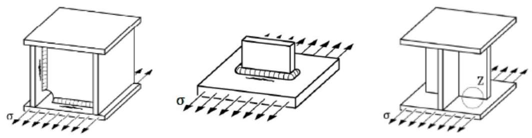
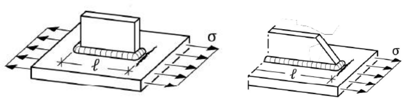
***Note**: Fatigue verification of HFMI-treated welds of vertical stiffeners to web in beams, can be made with reference to the principal stress as specified in EN 1993-1-9 for the as-welded situation.*

3.3 Verification of maximum allowable stresses

To ensure the stability of the beneficial compressive residual stresses generated by the HFMI-process, a verification of maximum allowable stress at each HFMI-treated detail should be performed.

The limitations for maximum allowable stress for all three details covered by this document is given in Table 2. Load effects for this verification should be obtained from the characteristic load combination of actions, including stresses from permanent, wind, thermal, and traffic loads. For the latter, LM1 and LM71 should be used for road and railway bridges respectively.

Table 2: Maximum allowable stresses for HFMI-treated welds

Detail	Limits
	$-0.9f_y \leq \sigma_{max} \leq f_y$
	$-0.7f_y \leq \sigma_{max} \leq f_y$
	$-0.5f_y \leq \sigma_{max} \leq f_y$

***Note**: No effect from HFMI-treatment can be accounted for if the limits presented in this section are not satisfied.*

***Note**: If HFMI-treatment is done in the workshop, it should be ensured that, after treatment, the structure is not treated in any way that results in exceeding the limits given in Table 2 (e.g. during handling, transportation, or any other operation such as launching).*

3.4 Verification of details in existing bridges

HFMI-treatment can be used to extend the fatigue life of existing bridges if the requirements and method of application stipulated in Section 4.1 are fulfilled.

For the purpose of calculating the remaining fatigue life of a welded detail after treatment, the calculation principles outlined in this document apply. In other words, treated welds in existing bridge can be treated as new HFMI-treated welds in fatigue verification.

In this case, the self-weight stress does not influence the treatment and Φ should be taken as zero.

It should be emphasized that HFMI is a local treatment method, the effect of which is confined to the very local area subject of treatment, i.e. weld toe. While fatigue cracking from weld toe is the governing failure mode in most welded details, extending the fatigue life with respect to this failure mode might result in other cracking modes or locations becoming decisive and thus limit the beneficial effect of the local toe treatment.

***Note**: When considering the use of HFMI for fatigue life extension of existing structures, the possibility of crack initiation at other locations, or at the same location but from other types of discontinuity (weld root, for example) must always be considered and assessed.*

***Note**: If Inspection reveals signs of shallow fatigue cracks or other surface defects, TIG-dressing can be used to remove (fuse) these surface defects, and HFMI-treatment can then follow TIG-dressing to maximize the gain in term of life extension. An extended fatigue life equivalent to that of a new HFMI-treated weld can be assumed.*

4. Technical requirements and quality assurance

4.1 Requirements on welds before HFMI-treatment

Good accessibility to the detail to be HFMI-treated is very essential for achieving an HFMI-treatment with good quality. This will depend (partly) on the HFMI-tool used.

The general requisite on welded details prior to HFMI-treatment is that they should meet the requirements for B-quality level in accordance with EN 1090-2 and ISO 5817.

In particular, the weld-quality measures shown in Figure 5 are essential for reaching the right quality of HFMI-treatment.

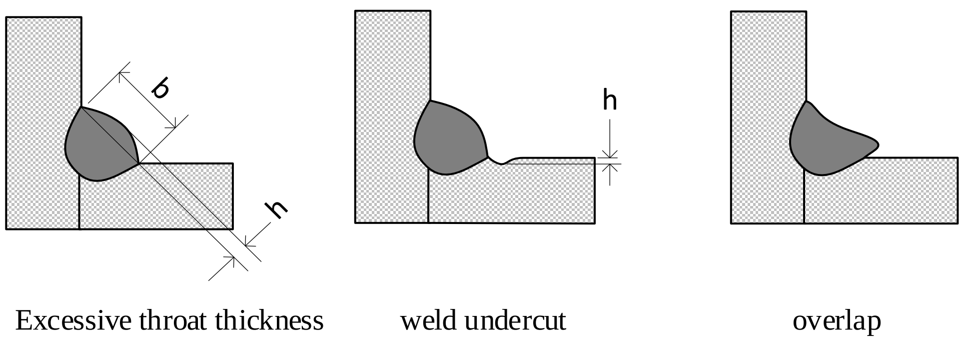


Figure 5: Weld quality measures that are most relevant for HFMI-treated welds

Moreover, the weld fusion line (i.e. weld toe) should be clear and visible. All foreign materials existing on the weld surface such as spatter, scale, oxides shall be de-slagged and removed before HFMI-treatment.

Any other operation that alters the weld toe geometry such as grinding and TIG-remelting is not allowed prior to HFMI-treatment.

***Note**: When HFMI is used for the purpose of fatigue life extension of existing welds, TIG-remelting can be used to remove existing shallow cracks or crack-like defects. Subsequently, HFMI can be applied on the "new" weld toe, preferably with larger pin diameter (e.g. Φ10)*

Light grinding may be used to achieve the required weld quality. It should however be performed with care to ensure that the weld line (the weld toe to be treated) is still visible after grinding. Therefore, it is recommended that grinding – if needed - is made under the supervision of an expert HFMI operator.

Sandblasting can be used before and after HFMI-treatment.

Inspection of the weld line before HFMI-treatment may reveal defect that need to be treated/corrected prior to application of HFMI-treatment, see Figure 6.

If the treatment is to be applied on welds in existing structures (i.e. structures in service) as a method of extending the fatigue life, the weld should also be inspected for surface cracks at weld toe prior to treatment (extent of inspection: 100%). HFMI-treatment of welds containing surface (toe) cracks is not recommended, even though extensive test results exist that verify that fatigue life extension is achieved even in the presence of surface cracks that are less than 2 mm deep.

4.2 Requirements for welds after HFMI treatment

The performance of HFMI-treated welds depends to a large extent on the compressive residual stresses generated locally at the treated weld toe. Therefore, any operation that might influence the state of residual stress in treated welds should strictly be prohibited after HFMI-treatment. Examples of permitted and prohibited operations on welds after HFMI-treatment are:

- All forms of thermal treatment are not allowed on HFMI-treated details
- Hot dip galvanization is not permitted
- If HFMI-treatment is done in the workshop, it should be ensured that, after treatment, the structure is not treated in any way (e.g. during handling, transportation or any other operation such as launching) that results in exceeding the limits given in Table 2.
- Sandblasting can be used to prepare the surface for corrosion protection after HFMI-treatment.
- Sharp groove edges that could promote corrosion can be removed by light grinding before applying the corrosion protection, see Figure 6.

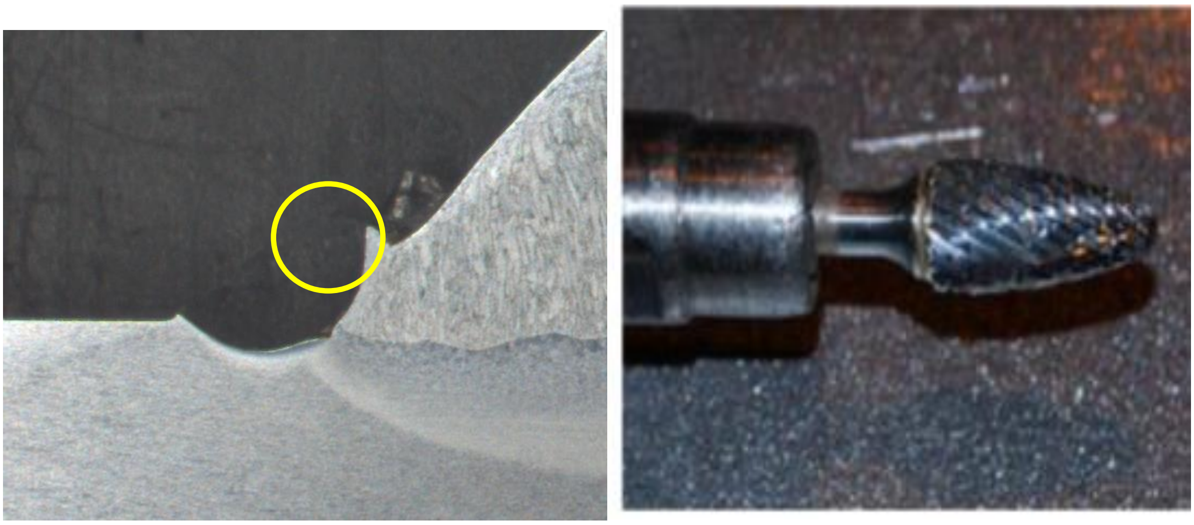


Figure 6: Sharp groove edges that may result after HFMI-treatment may be removed by light grinding prior to application of corrosion protection coating. Use a burr grinder for this purpose.

4.3 Personnel requirements and quality control

A well-performed good-quality HFMI-treatment relies to a large extent on the experience and competence of the operator.

HFMI-treatments should only be performed by trained and qualified operators. The qualification of operators should be based on suitable training related to the device manufacturer and is not transferable to other devices.

The HFMI-tool operator and quality inspection personnel should have theoretical knowledge about HFMI treatment, fatigue of welds and common weld imperfections and defects.

For qualified technologies, a visual inspection of HFMI-treatment (trace of indentation) should be carried out by the operator and confirmed by a supervisor or inspector (extent of inspection: 100%).

Both quantitative and qualitative measures of the quality of HFMI-treatment are given below. These should also be documented, see Section 5.

Treatment of the weld toe should result in an indentation (groove) which is uniform and smooth. The depth of the indentation is the most important indicator of a successful HFMI-treatment. Dependent on the strength of the steel and the size of the indenter used, an optimum value for the depth of the groove is 0,25-0,5 mm. Both undertreatment and overtreatment should not be accepted. The depth of the groove can be verified using simple gauges, which should be done at regular intervals along the weld line, see Figure 8. More advanced methods, such as laser scanners, are also available.

The radius of the groove depends on the radius of the pin used. However, it is essential to verify that the treatment width (w) covers both the weld material and the base metal, see Figure 7. Typically, this width is 2-5 mm, and at least 25% of this width should be at either side of the weld fusion line.

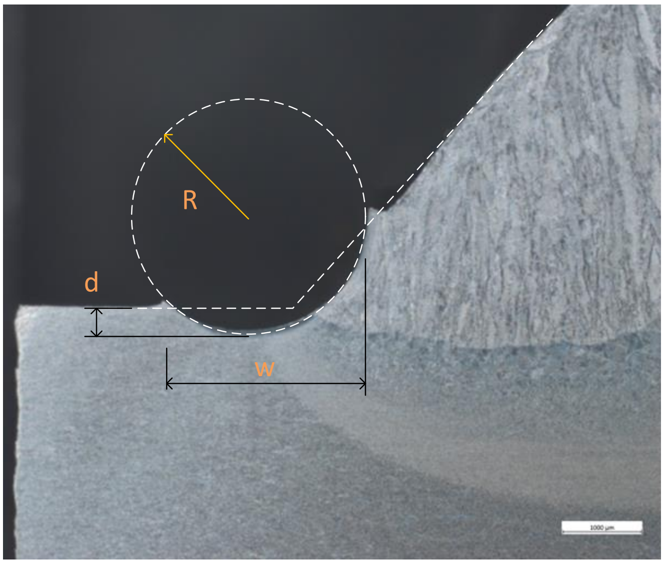


Figure 7: Measures of a ssuccessful HFMI-treatment of a weld toe

Figure 8: Simple gauge for measuring the depth of the HFMI-groove

Visual inspection, assisted by magnification glass, provide additional measures for more qualitative quality assurance

- The surface of the HFMI-groove should be smooth and shiny without clear distinct marks of the individual indenter strikes.
- It should be ensured that relevant weld toes have been completely treated and that the original weld fusion line has been entirely plastically deformed. No remaining crack-like lines should be visible in the groove, see Figure 9.
- Inspection of the HFMI groove after treatment might reveal shallow, sub-surface defects (e.g. porosity), see Figure 10. In such case, the weld should be repaired (e.g. by gauging and re-welding) and the new weld line re-treated.

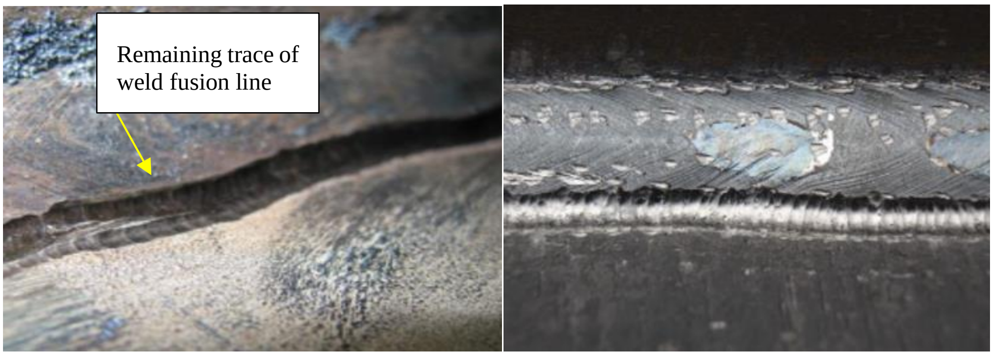


Figure 9: Examples of properly and improperly treated welds. The latter is characterized here by a trace of the weld fusion line remaining in the HFMI-groove. The HFMI-groves are all with smooth and shiny surface without marks of the individual indenter strikes.

Figure 10: HFMI-treatment revealing shallow, sub-surface porosity. Also visible are surface (possibly cold) cracks at weld toe. Such weld should be repaired prior to HFMI-treatment.

4.4 Requirements particular for welded details in existing bridges

The following should be noted when HFMI-treatment is applied to extend the fatigue life of welded details in existing bridges.

1. The use of HFMI-treatment on existing bridges should be limited to steel types that are equivalent to modern steels in term of strength and ductility.

2. HFMI applied for the purpose of fatigue life extension of existing structures should always be preceded by proper inspection including NDT testing. The objectives of inspection could be to:
   a. verify that the weld to be treated fulfil the requirement for weld quality, as specified for the method of treatment. For example, quality level B is required prior to application of HFMI-treatment.
   b. verify that the welds to be treated are free from fatigue cracks. If cracks are detected an appropriate NDT testing should be employed to accurately quantify crack dimensions (length & depth).

***Note**: HFMI-treatment may only be applied on existing welded details after verification that these details are crack-free.*

***Note**: If Inspection reveals signs of fatigue cracking, an appropriate NDT testing should be used to determine the crack dimensions with good level of accuracy and reliability. Shallow cracks (up to 3 mm deep) can be fused by TIG-dressing or removed by grinding. If TIG is used, the treatment can be proceeded with HFMI treatment to maximize the gain of treatment. An extended fatigue life equivalent to that of a new HFMI-treated weld can be assumed in this case.*

3. If the conditions above are fulfilled, the use of HFMI has been verified to result in “erasing” fatigue damage that has accumulated under the service life of the bridge. It should be pointed out, however, that the beneficial effects obtained from HFMI-treatment are confined to the very local region at the weld toe. Other cracking modes or locations might therefore become decisive. This is illustrated in Figure 11, where in (A), after treatment of weld toe at the end of the cover plate detail, cracking initiates from weld root; and in (B) the continuous weld takes over the risk for fatigue cracking.

***Note**: When considering the use of improvement methods for fatigue life extension in existing structures, the possibility of crack initiation at other locations must always be considered and assessed.*

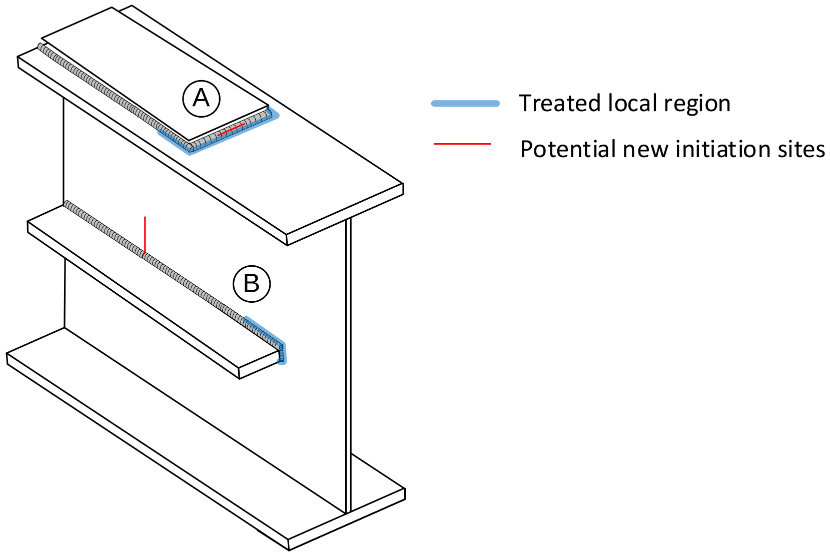


Figure 11: Other cracking modes/locations might need to be checked when HFMI-treatment is applied to extend the fatigue life of existing welded details with respect to toe cracking.

5. Documentation

A complete documentation of HFMI-treatment should typically comprise information provided by designer or client, operator, and inspector.

The following information should be provided to the HFMI-operator and inspector <u>by the designer or the client</u>

- Detailed drawings showing the location and extent (e.g. length) of HFMI-treatment to be performed
- Information on plate thickness, weld type and weld size, if not specified on the drawing
- Information about steel grade and weld material (strength)
- Specification of weld quality (level B is required). Weld quality should – in particular – be verified and documented after production of members or details to be treated.
- If treatment is applied for the purpose of fatigue life extension of existing welded details, NDT tests should be conducted to verify that the welds are free from surface cracks (at weld toe)
- Early discussion with HFMI-provider or expert can be very valuable to ensure that the intended treatment can be performed to the right quality, e.g. with reference to access to the detail to be treated (this will often depend on the HFMI-provider, tool used, etc.)

The following information should be documented <u>by the operator</u> of HFMI-tool:

1. Operator name
2. Date/time
3. Remarks (if any) from inspection of weld before treatment
4. Tool model
5. Impact frequency
6. Impact amplitude
7. Indenter diameter
8. Travel speed
9. Number of passes, if more than one.

Other remarks (e.g. possible operation done on weld before treatment, treatment interruption and re-start if any, etc.)

Possible defects or deviations from required weld quality measures that are detected by the operator should be reported and corrected before proceeding with HFMI-treatment.

After HFMI-treatment, an inspector should inspect, document, and approve the HFMI-Treatment. The inspection should be conducted by an expert other than the operator. Documentation produced <u>by the inspector</u> include:

1. Verification of the treated extent/length in accordance with the drawings
2. Comments and remarks from visual inspection, see Section 4.3
3. Documentation of the geometry of the HFMI-grove, in particular groove depth & width
4. Photographs of the treated welds, before and after treatment

6. Design example

In this section, an example covering fatigue verification of an HFMI-treated structural detail in a composite road bridge is given. The bridge is simply supported with a span of 32.0 m. The detail considered is a welded vertical stiffener to the lower flange of a bridge girder (i.e. non-load carrying transverse welded attachment), see Figure 12. The bridge design life is 80 years, and the bridge is made of S690 structural steel. The design should be made considering 'safe life approach' with high consequence of failure.

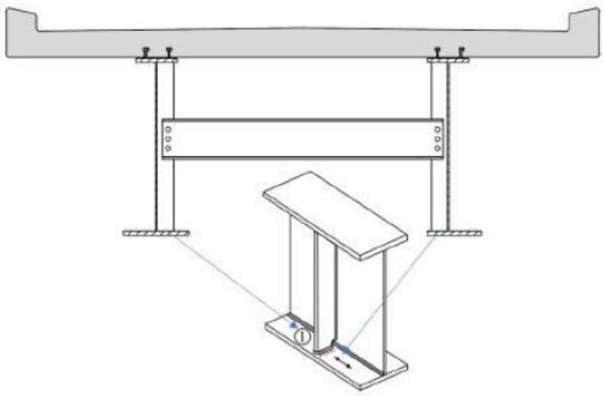


Figure 12: HFMI-treated welded detail for fatigue verification in road bridge

Tables 3 and 4 summarize the relevant bridge data. The section modulus in the mid-span at the location of the detail to be checked is 3.6×10^{7} mm^3. Traffic category '4' is used for design, which indicates the passage of 50 000 lorries in the slow lane per year. The average weight of the lorry for regional traffic is assumed to Q_{M1} = 310 kN. The reference value Q_0 is equal to 480 kN, and the reference number of lorries passing over the slow lane is N_0 = 500 000.

The stress from permanent load at the location of the detail to be checked is equal to 120 MPa. The fatigue life of the structural detail is to be assessed using both the λ-coefficients method together with fatigue load model FLM3, and the damage accumulation method using FLM4. A load distribution factor of 0.833 is used for both fatigue load models as the truck in both models have the same width.

Table 3: Bridge data for the calculation example

Material yield stress	f_y [MPa]	690
Bridge beam section modulus	W [mm^3]	3.6×10^{7}
Stress from permanent load on detail	σ_{perm} [MPa]	120
Partial factor on load	γ_{Ff}	1.0
Partial factor on resistance	γ_{Mf}	1.35
Design life	Years	80
N_{obs}	Cycles	50 000

The reference fatigue strength of the HFMI-treated detail (see Table 1) and the corresponding points defining the fatigue strength are given in Table 4. Similar data is given for the base metal.

Table 4: Detail categories for base metal and HFMI-treated detail at the control section

Base metal S-N curve data		HFMI-treated detail S-N curve data	
$\Delta\sigma_{C,BM}$	160.0	$\Delta\sigma_{\mathrm{C,HFMI,ref}}$	140.0
$\Delta\sigma_{D,BM}$	117.9	$\Delta\sigma_{D,\mathrm{HFMI,ref}}$	116.6
$\Delta\sigma_{L,BM}$	64.7	$\Delta\sigma_{L,\mathrm{HFMI,ref}}$	83.6
		$\Delta\sigma_{S}$	324.1

For the HFMI-treated detail:

$$\Delta\sigma_{\mathrm{C,HFMI,ref}} = 140\ \mathrm{MPa}$$

$$\sigma_{\mathrm{D,HFMI}} = 0.833 \times \Delta\sigma_{\mathrm{C,HFMI}} = 116.6\ \mathrm{MPa}$$

$$\Delta\sigma_{\mathrm{L,HFMI}} = 0.717 \times \Delta\sigma_{\mathrm{D,HFMI}} = 83.6\ \mathrm{MPa}$$

$$\Delta\sigma_S = \left(\frac{\Delta\sigma_{C,HFMI}{}^5}{\Delta\sigma_C^3}\right)^{0.5} = 324.1\ \mathrm{MPa}$$

The correction factor for yield strength is:

$$\mathrm{f_1} = 1 + \frac{0.1\ (\mathrm{f_y} - 355)}{\Delta\sigma_{\mathrm{C,HFMI,ref}}} = 1 + \frac{0.1\ (690 - 355)}{140} = 1{,}24$$

6.1 Verification using the simplified λ-coefficient method

The passage of FLM3 over the influence line for bending moment at the location of the considered detail results in a maximum bending moment and a stress range of :

M_{max} = *2976 kNm* (*Same as* ΔM as the bridge is simply supported, M_{min} =0).

$\Delta\sigma_E = \Delta\sigma_P$ = *82.7 MPa*

The λ-coefficients are listed in Table 3

Table 3: λ-coefficients for fatigue verification with FLM3.

λ coefficient	***Takes into account***	***Value***
λ_1	*Bridge length*	*2.55 -0.7(L-70)/70 =* ***2.33***
λ_2	*Actual traffic flow*	$(Q_{M1}/Q_0).\ (N_{obs}/N_0)^{1/5}$ = ***0.407***
λ_3	*Design fatigue life*	$(t/100)^{1/5}$= ***0.956***
λ_4	*Interaction of lanes*	***1*** *(for single laned traffic)*
λ_{max}	*Maximum* λ value	***2*** *(for bridge longer than 25 m)*
λ	*Damage equivalent factor*	$\lambda_1\ \lambda_2\ \lambda_3\ \lambda_4 < \lambda_{max}$ = **0.907**

Since the HFMI-treatment is performed in the workshop, the effect of R-ratio (or mean stress) should be considered through the parameter λ_{HFMI}. For road bridges:

$$\lambda_{HFMI} = \frac{2.38\Phi+0.64}{\Phi+0,66} = \frac{2.38.\left(\frac{120}{2\times 82.7}\right)+0.64}{\left(\frac{120}{2\times 82.7}\right)+0.66} = 1,71$$

With Φ calculated as given in eq (19) in Section 3.2.3.

The fatigue verification for the λ-coefficients method reads:

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

with

$$\Delta\sigma_{\mathrm{e,2,HFMI,Ed}} = \lambda \times \lambda_{HFMI}\ \Delta\sigma_{\mathrm{E}}\gamma_{Ff}$$

$$\Delta\sigma_{\mathrm{e,2,HFMI,Ed}} = 0.907 \times 1.71 \times 82.7 \times 1.0 = 128\ \mathrm{MPa}$$

$$\mathrm{f_1} \times \Delta\sigma_{\mathrm{HFMI,C,ref}} = 1.24 \times 140 = 173.6\ \mathrm{MPa}$$

and the verification reads:

$$\frac{128}{173.6/1.35} = 0.99 < 1.0$$

As the fatigue strength obtained for the detail is greater of that for the base metal (173.6 vs 160), the base metal should also be checked as follow:

$$\frac{\lambda \Delta\sigma_{\mathrm{E}} \gamma_{Ff}}{\Delta\sigma_{C,BM}} = \frac{0.907 \times 82.7 \times 1.0}{160/1.35} = 0.63 < 1.0$$

Note: If HFMI-treatment is to be applied on-site (after bridge erection), Φ is taken as zero and λ_{HFMI} becomes 1.0. Thus, $\Delta\sigma_{\mathrm{e2,HFMI,Ed}}$ becomes 74.8MPa. The verification of the treated weld toe would then read:

$$\frac{74.8}{140/1.35} = 0.72 < 1.0$$

The base metal verification is the same as above.

6.2 Verification using the method of damage accumulation

Conducting the verification of a HFMI treated detail using the damage accumulation method & FLM4 requires calculating the equivalent stress range. In this example, local traffic is assumed which indicates that lorry type 1 of the set of FLM4 represents 80% of the total number of lorries, and the remaining 4 lorries in the set represents 5% of the traffic each.

System analysis results in the load effect values given in Table 4.

Table 4: Maximum moments and stress ranges at the location of the studied detail due to the passage of different vehicles of FLM4.

Load	*M_{max} (kN.m)*	*Stress range (MPa)*
FLM4 (Lorry 1)	*1443*	$\Delta\sigma_1 = 40$
FLM4 (Lorry 2)	*2255*	$\Delta\sigma_2 = 63$
FLM4 (Lorry 3)	*3061*	$\Delta\sigma_3 = 85$
FLM4 (Lorry 4)	*2380*	$\Delta\sigma_4 = 66$
FLM4 (Lorry 5)	*2668*	$\Delta\sigma_5 = 74$

The stresses from FLM4 should be compared to the reference points of the S-N curve:

$$\frac{\Delta\sigma_{D,\mathrm{HFMI,ref}}}{\gamma_{\mathrm{Mf}}} = 86\ \mathrm{MPa}$$

$$\frac{\Delta\sigma_{L,\mathrm{HFMI,ref}}}{\gamma_{\mathrm{Mf}}} = 62\ \mathrm{MPa}$$

The stress from lorry type 1 is below the cut-off limit and can, therefore, be neglected. All other stress ranges fall on the part of the S-N curve with slope of 9.

None of the stress ranges in Table 4 is above the limit stress range for accounting for the beneficial effect of HFMI-treatment:

$$\frac{\Delta\sigma_S}{\gamma_{\mathrm{Mf}}} = \frac{\left(\frac{140^5}{80^3}\right)^{0.5}}{\gamma_{\mathrm{Mf}}} = 324\ \mathrm{MPa}$$

The equivalent stress can be calculated from equation (28)

$$\sigma_{\mathrm{eq}} = \sqrt[9]{\frac{\sum_{j=1}^{4}(n_j \times \Delta\sigma_j^9)}{\Sigma n_j}} = \sqrt[9]{\frac{(2500\times 63^9 + 2500\times 85^9 + 2500\times 66^9 + 2500\times 74^9)}{50{,}000}} = 63.5\ MPa$$

$$N_{eq} = 5.10^6 \left(\frac{f_1\ .\ \Delta\sigma_{D,HFMI.ref}\ /\gamma_{Mf}}{\lambda_{HFMI}.\ \Delta\sigma_{eq}.\gamma_{Ff}}\right)^{m_2} = 5.10^6 \left(\frac{\frac{1.24\times\ 116.6}{1.35}}{1.71\times 63.5\times 1.0}\right)^9 = 4.4\times 10^6\ cycles$$

$$D = \frac{n}{N_{EQV}} = \frac{50{,}000 \times 80}{4.4 \times 10^6} = 0.9$$

The endurance of the base metal is calculated as follow:

$$N_{eq} = 5.10^6 \left(\frac{\frac{\Delta\sigma_{D,metal}}{\gamma_{Mf}}}{\Delta\sigma_{eq}.\gamma_{Ff}} \right)^5 = 5.10^6 \left(\frac{\frac{160 \times 0.737}{1.35}}{63.5 \times 1} \right)^5 = 24.6 \times 10^6 \; cycles$$

$$D = \frac{n}{N_{EQV}} = \frac{50{,}000 \times 80}{24.6 \times 10^6} = 0.16$$

6.3 Verification of maximum allowable stress

Finally, the allowable stresses should be checked using the characteristic load combination. As the details is only subjected to tensile load cycle, the maximum stress should be verified to be less than $+1\times f_y$. This includes the self-weight, the shrinkage-induced stresses, the stress induced by the traffic load model, LM1 including both the concentrated force (TS), and the distributed load (UDL), and the environmental loads (wind, F_w and temperature T_k).

$$SW + (1 \; or \; 0) \times S + TS + UDL + 0.6 \times \max(F_w, T_k) = 300 \; MPa = 0.45 f_y$$

7. References

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