Fatigue assessment and retrofitting of Vilvoorde viaduct in Belgium

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Abstract

The paper presents the fatigue assessment and subsequent design of reinforcement of the Vilvoorde viaduct in Belgium. The project aims to extend the service life of the viaduct, built in 1978, until 2078 with additional lanes of heavy traffic. The calculation strategy developed specifically for this project, combining Eurocode S-N curves, fracture mechanics and inspections, as well as the implemented reinforcements, are presented.

Keywords: Fatigue assessment; Fatigue reinforcement; Fracture mechanics

1 Introduction

The 'Vilvoorde viaduct' (BE) (Figure 1) is a major engineering structure located on the northern section of the Brussels ring road. It consists of 4 x 2 parallel viaducts varying in width from 20 m to 27 m. Two of these are composite bridges, while the main viaducts (section B in Figure 2) have an orthotropic slab and a central steel box 8 m wide and 5.5 m high (cf. Figure 2). They are made up of seven spans varying between 93 m and 162 m. In service since 1978 with three lanes of traffic, the viaduct now presents risks of fatigue, mainly in its orthotropic slab and obvious signs of deterioration in the concrete of the slab of the composite decks. A renovation of the viaduct is thus undertaken to extend its service life to 2078, and to enable it to accommodate up to 4 lanes of traffic, including 2 'heavy' lanes on each viaduct. As part the renovation, Greisch carried out retrofitting and fatigue studies of the main steel viaducts.

Figure 1: View of Vilvoorde viaduct

Some aspects made the study particularly complex:

- The general constraints related to an existing structure (accessibility, existing material, …)
- The viaduct has already suffered damage from past traffic. The presence of this damage cannot be overlooked, yet it is not easy to quantify. In fact, damage is not always manifested by the presence of a crack, or at least a crack large enough to be detected.

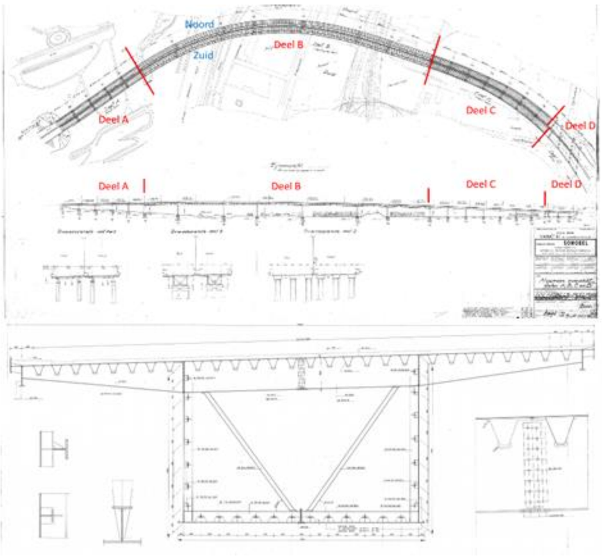

Figure 2: Location and cross section of section B

- A highly detailed calculation model was required. This is because, in a deck with an orthotropic slab, a large proportion of the damage is due to local effects associated with the passage of truck wheels (the dimensions of the slabs components do not match today's standard recommendations of the Eurocode).

- The specifications imposed the analysis of a large number of scenarios.

2 Traffic hypotheses

In this specific case, fatigue calculations demand to consider the whole life of the bridge (see further): past (1978-2028) and future (2028-2078)

2.1 Past (1978-2028) - Periods 1 to 6

Past is subdivided into six different periods, to account for:

- Changes in lanes over time
- Changes in trucks type over time
- Increase in number of trucks over time
- Increase in truck weight over time

For each period, only one heavy traffic lane was present (no overtaking allowed).

2.2 Future (2028-2078) – Period 7

For the future, two possible scenarios must be considered, with the possibility to switch from one to the other over time (see Figure 3):

- 7a - 3 traffic lanes: one heavy lane, inside the box girder (100% of occurrences)
- 7b - 4 traffic lanes:
    - one single heavy lane on the cantilever (80% of occurrences)
    - heavy lane on the cantilever + overtaking lane (20% of occurrences).

In period 7b, two identical trucks of the same weight, side by side, are considered when

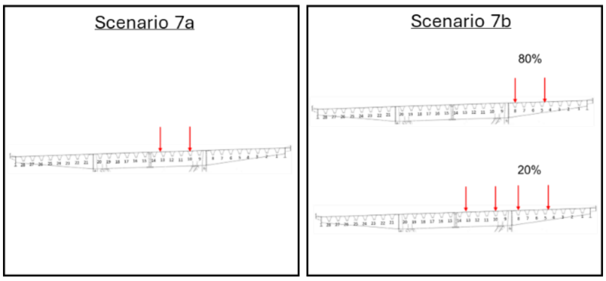

Figure 3: Future traffic scenarios

overtaking must be taken into account. Finally, in practice future as well is subdivided into five (sub)periods of 10 years each, to account for an increase in trucks weight (total increase of 20% over 50 years, between 2028 and 2078).

3 Structural model and fatigue loads

Although transversal behaviour of the bridge deck is mainly governed by local effects, longitudinal stresses result from both local and global effects. To correctly account for all the possible contributions to the stress field a global and local model are superposed (cf. Figure 4). Both models are realized with FINELG, Greisch in-house finite element (FE) solver, developed in collaboration

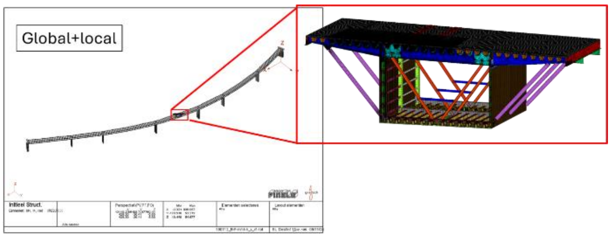

Figure 4: Global+local FE model

with the University of Liège. The global model is a beam model representing the whole viaduct. Concerning fatigue traffic loads, four different convoys must be considered at the global level (cf. Figure 5). These configurations must be evaluated for each truck type applied on the local model (see further).

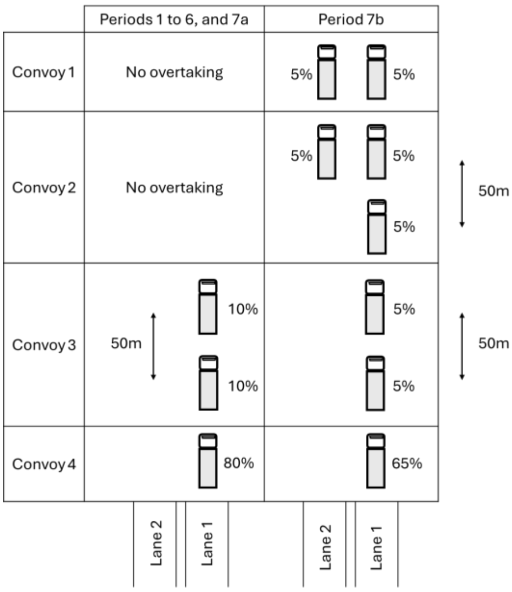

Figure 5: Global convoys definition

The local model is a shell/volume model representing 16m of viaduct (five frames, cf. Figure 4 and Figure 6). This model obviously differs between past (no reinforcements) and future (reinforced structure, see ch.7), so that in reality two different local models are needed. Traffic loads

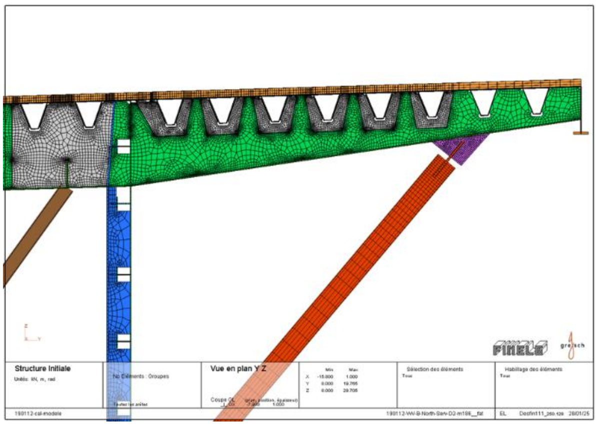

Figure 6: Detail of the local model (at midspan)

on the local model consist of up to 6 different trucks (all present in the future, not all present in the past), as depicted in Figure 7, each one of them

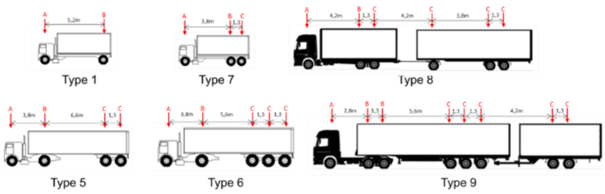

Figure 7: Type of trucks to consider for local loads

subdivided into three weight classes (light, medium, heavy). For the evaluation of local effects only, an uncertainty on the transversal position of the trucks is also considered. Therefore, for each lane five different transverse positions are considered, each with a certain probability of occurrence (cf. Fig. 4.6 in [1]).

Overall, 642 FE simulations would be needed to cover all the possible scenarios (past+future). To reduce the computational burden, only unitary axles (with their geometries) are simulated and the actual truck passages simulated by recombining unitary influence lines using TRAFALI, an in-house post-processing software. Doing so, the number of FE simulations needed is reduced to 91 (including both past and future) and the 930 stress histories resulting from the different trucks/scenarios are thus reconstructed in post-processing. According to the calculation approach described in the following paragraph note that, once the stress histories are known, it is still necessary to:

- Compute past (D_1) and future (D_2) damage for 134 details, at different locations on the structure, for a total of ~4,500 check points.
- For those details which cannot be justified according to a safe-life approach ($D_1 + D_2 > 1$), apply a damage tolerant approach by computing crack propagation curves (see 4.2).

4 Calculation approach

Since the study concerns an existing structure, several considerations must be made:

- the structure is already damaged, and therefore cracks can already be present;
- These cracks may or may not be visible (due to their position and/or size);
- It cannot be assumed that if a crack is not detected, it is not present. In other words, even if no cracks are detected on a particular detail, it would be wrong to perform a classical fatigue verification based on S-N curves [2] from today to the end of service life (i.e. 2078), considering the detail as new (current damage (D_1) = 0).

There are three reasons for this:

- A crack size, a, on a given component can be 'critical', i.e. once it reaches such a limit, the crack becomes unstable and propagates "indefinitely", possibly leading to the component failure. The critical size depends, amongst other, on the loading applied to the crack. It may be very well possible that a potential crack exists in the structure (smaller than the sensitivity of the measuring instrument at hand) and that, if the applied load is large enough, its size is already critical.
- Fatigue loads can lead to the propagation of an existing crack (which may not be critical in the first place). The crack size will therefore increase over time and potentially become critical.
- Cracks are initiated by a certain number of stress cycles which produce no visible damage.

It is important to highlight that many details on the existing structure already showed a computed past damage larger than one. Thus, if only a safe-life approach were to be used, it would have been simply impossible to justify the bridge. Hence, to be able to proceed with the renovation and avoid the demolition of the bridge, it was decided to deviate from the Belgian national annex to EN 1993-1-9 and employ a damage tolerant approach when needed. This leads to a “hybrid” strategy, based on both Eurocode S-N curves and fracture mechanics, which is detailed in the following.

4.1 Some words on fracture mechanics

Before going further, we spend few words on fracture mechanics, since it is seldomly exploited in civil engineering. Fracture mechanics describes the propagation of cracks in a structure or component. In our case, the final goal is to establish, for each crack potentially present on the bridge:

- Its critical size, a_{crit};
- Its propagation curve under cyclic traffic loading from the end of the works until the end of service life (2078) (see e.g. Figure 8).

Both quantities depend on a series of parameters:

- Stress variations histogram at the location of the crack for the period going from the end of the works to 2078;
- Geometry of the crack (through-thickness or surface crack);
- SLS frequent stresses at the crack location;
- The presence (or not) of a weld near the crack;
- Steel toughness.

Other factors, such as the stress state at crack location, or the degree of hyperstaticity, can influence crack propagation, but they were not considered here. To compute the propagation curve, *Paris law* is employed:

$$\frac{da}{dN} = C\big(\Delta K(a)\big)^m$$

This law establishes that crack size variation with respect to the number of cycles, N, is proportional to the stress intensity factor variation, $\Delta K(a)$, to the power m, where C and m are two constant material-dependent parameters. An example of double-slope Paris law, as proposed in [4] and employed in this project, is given in Figure 9.

Stress intensity factor variation is defined as follows:

$$\Delta K = Y(a, \dots)\sqrt{2\pi a}\Delta\sigma$$

Where $\Delta\sigma$ is the stress variation at the crack location, typically induced by traffic loading, and Y is a corrective factor which depends on the specific geometry of the component and on the crack size, a. Note that ΔK depends on a (directly and via Y), which means that crack propagation computation needs an iterative procedure. For the same reason, the order of application of the different stress cycles is important (oppositely to safe-life approach). However, for practical reasons, in the present case only the total histogram of stress variation (over the future service life) is computed.

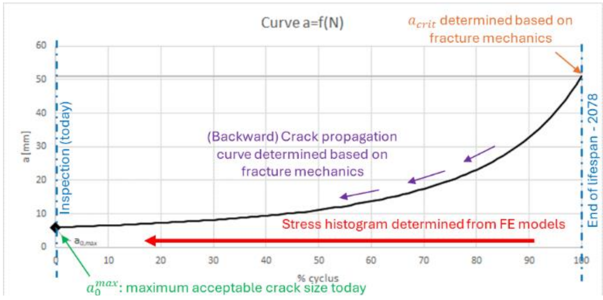

Figure 8: Example of crack propagation curve

To mimic the fact that in reality there is an alternance of different stress levels along time, the total histogram is subdivided into 100 sub-histograms which are subsequentially applied to compute crack propagation. Each sub-histogram is thus representative of ~6 months of traffic (see Figure 9).

As schematized in Figure 8, the propagation calculation is carried out assuming that the crack size has reached its critical value, a_{crit}, at the end of the desired life of the structure (i.e. 2078). The

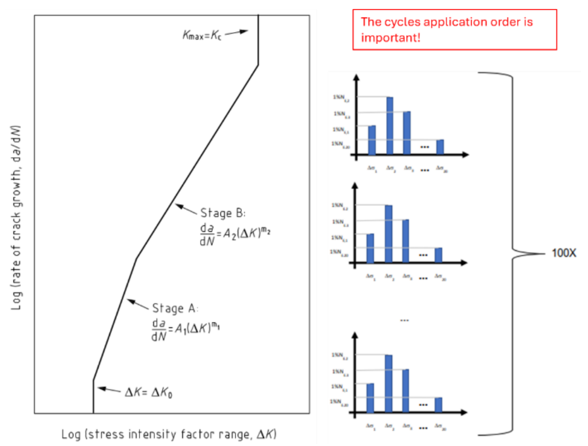

Figure 9: Double-slope Paris law and stress histograms used for crack propagation

calculation is then carried out "backwards", i.e. by applying stress variation (sub-)histograms in reverse order back to the time of the inspections (i.e. end of renovation works). The crack size obtained in such way is called a_0^{max}, and is the maximum acceptable crack size at the time of inspection. In other terms, this is the crack size that can be accepted today, so that in 2078 this same crack, after having propagated under fatigue loads, remains smaller than the critical crack size.

For more details, the interested reader can refer to [3]. Fracture mechanics calculations as performed in this project are largely based on this document.

4.2 General strategy

As already pointed out, a "hybrid" fatigue assessment is employed in this work, mixing classical fatigue verification (EN 1993-1-9 [2]) and fracture mechanics computations [3]. This approach is described in detail in the following.

For the sake of clarity, let us give some definitions:

- D_1: past damage for a given detail, calculated according to S-N curves [2], for the current life of the bridge (~50 years);
- D_2: future damage for given detail, calculated according to S-N curves [2], for the future desired service life extension (50 years);
- a_{crit}: critical crack size (in the sense of fracture mechanics), which depends on the static loads to which the structure is subjected (characteristic SLS), the material, particularly its toughness, and temperature;
- a_c: detectable crack size with sufficient probability *(usually tabulated values, depending on the inspection technique employed)*.
- a_0: crack size measured on the structure at time of the inspection (may be length or depth, depending on the type of crack);
- a_0^{max}: maximum acceptable crack size at the time of inspection (computed based on crack propagation curve).

The procedure is schematized in the workflow of Figure 10. For each fatigue detail:

- Compute the theoretical damage due to the past (from 1978 to 2028), D_1, according to S-N curves. This is done on the unreinforced structure.
- Compute the theoretical future damage (from 2028 to 2078), D_2, according to S-N curves. Conversely, this is done on the reinforced structure (see ch. 7).
- For all details, evaluate the total damage $D_{tot} = D_1 + D_2$.

There are three possible outcomes:

1. $D_{tot} < 1$: the detail is not critical at all. No action is required, not even inspection.
2. $D_2 > 1$: whatever the current state of the structure, the planned reinforcements are not sufficient, so the design must be modified. In the case of existing details, this may not be possible: future inspections must be foreseen.
3. $D_{tot} > 1$, but $D_2 < 1$: the reinforcements are sufficient, but a potential crack exists. Proceed with fracture mechanics calculations:
    - Compute the maximum crack size acceptable today, a_0^{max}.
    - If a_0^{max} is below the sensitivity of available measuring instruments, a_c:
        - If possible, the structure should be strengthened until a_0^{max} becomes measurable.
        - If not possible, due to the different constraints typical of an existing structure, future inspection intervals must be defined based on the value of a_c so that reliability is ensured.
    - Otherwise, if $a_0^{max} > a_c$, proceed with inspection:
        - If the measured crack size a_0 is greater than a_0^{max}, the crack must be repaired.
        - If the measured crack size a_0 is less than a_0^{max}, the crack can stay in place, without repairs.

The present approach allows to take into account the past life of the structure, and at the same time to limit the areas to be inspected and/or repaired.

5 Inspection techniques

Two techniques are employed for the inspection:

- Time Of Flight Diffraction (TOFD): for cracks in the deck-plate.
- Magnetic Particle Inspection (MPI): for all other details, both for surface and through-thickness cracks.

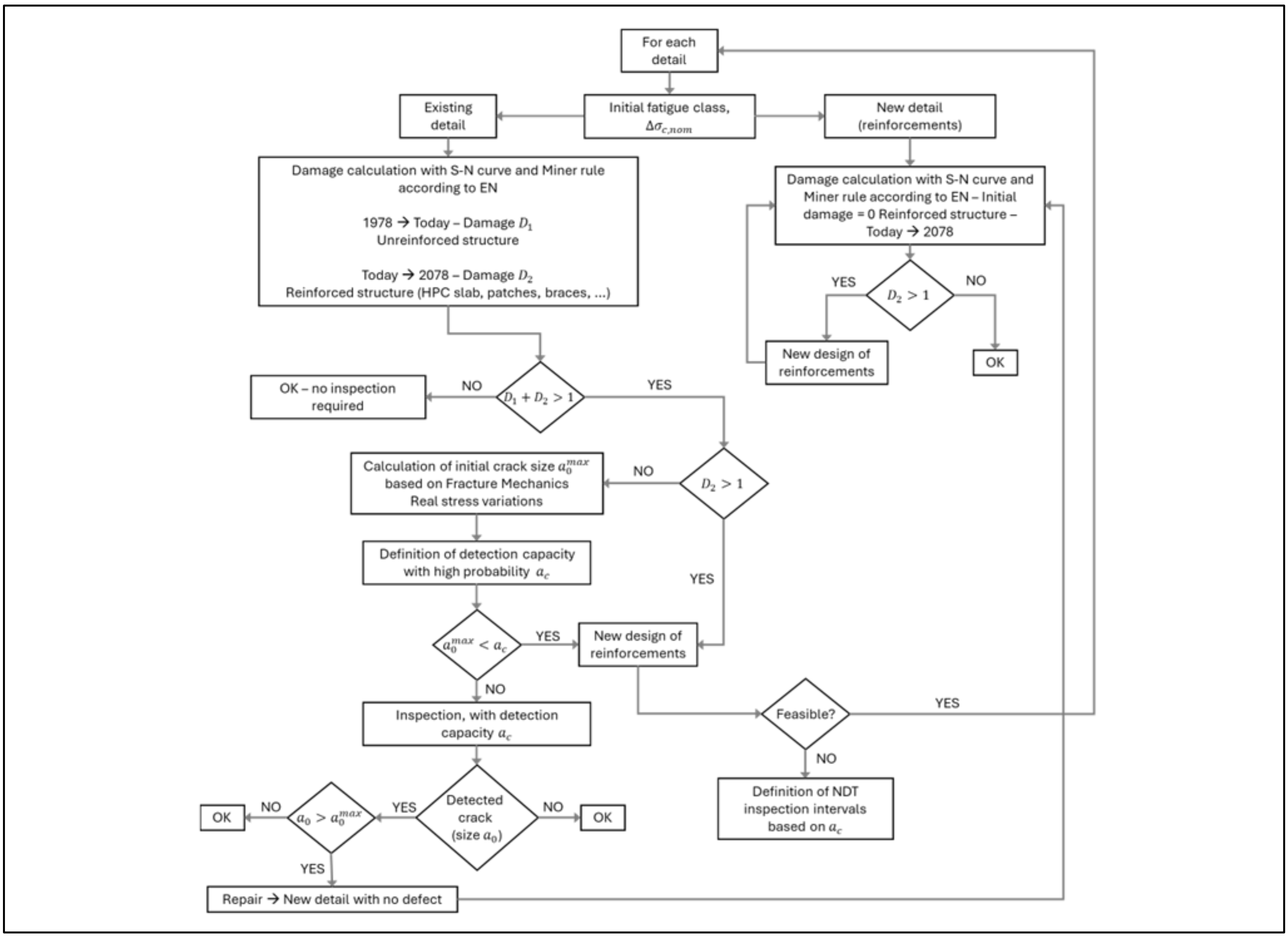

Figure 10: Fatigue analysis strategy for the renovation of Vilvoorde viaduct

6 Typical cracks

In the following a selection of typical cracks found on the viaduct are presented.

6.1 Deckplate

Cracks initiated at lower deck-plate face, at hard-points generated by u-ribs or crossbeam (cf. Figure 11).

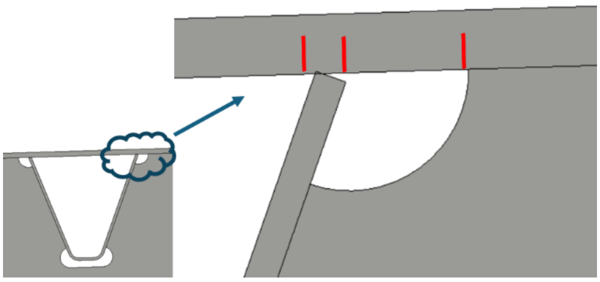

Figure 11: cracks at deck-plate lower face

These cracks can develop on a certain length (cf. Figure 12).

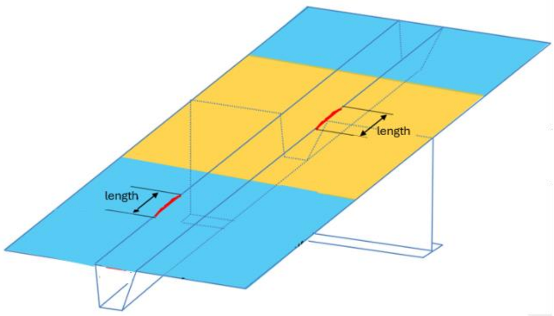

Figure 12: cracks in deck-plate can develop longitudinally on a certain length

6.2 Copeholes

Cracks developing from the cope-hole edge (Figure 13) due to the moment generated by the effects of shear between the deckplate and the web of the crossbeam.

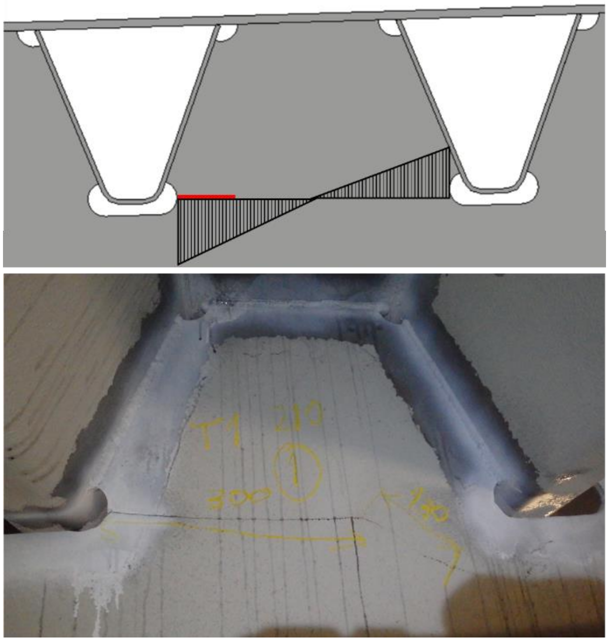

Figure 13: cracks developing from cope-holes

6.3 U-rib-deckplate welds

Crack at root of u-rib-crossbeam weld (Figure 14), due to both u-rib torsion and vertical loads under the passage of wheels.

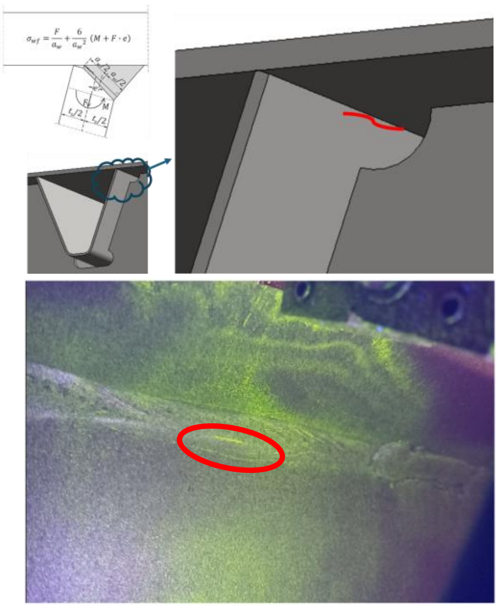

Figure 14: cracks at u-rib-deck-plate weld

6.4 U-rib-crossbeam welds

Crack developing at weld toe between the u-rib and the cross beam. Cracks are usually found in the bottom part of the connection. Depending on the location of the crack, this can be due either to shear transfer between the u-rib and the crossbeam (as in Figure 15), or to longitudinal stresses related to u-rib bending.

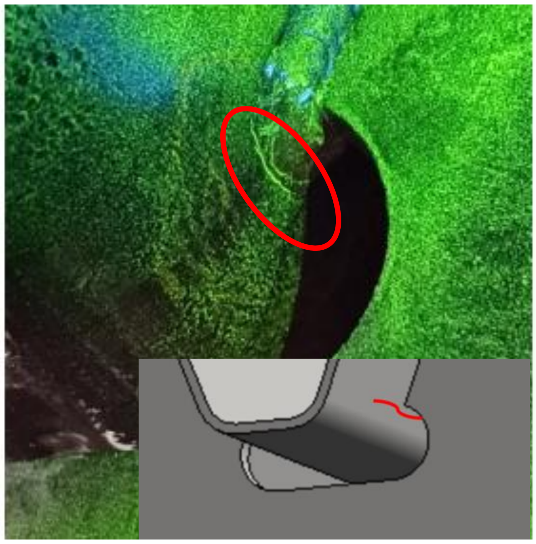

Figure 15: weld toe cracks at crossbeam-u-rib joint

6.5 U-ribs joints

Bolted u-rib joints are known to perform quite badly in fatigue. Calculations indicated that different kinds of cracks could appear in those connections, one of the most likely being the one in the upper cope-hole (cf. Figure 16). These cracks were actually found on the real structure, as shown in Figure 16.

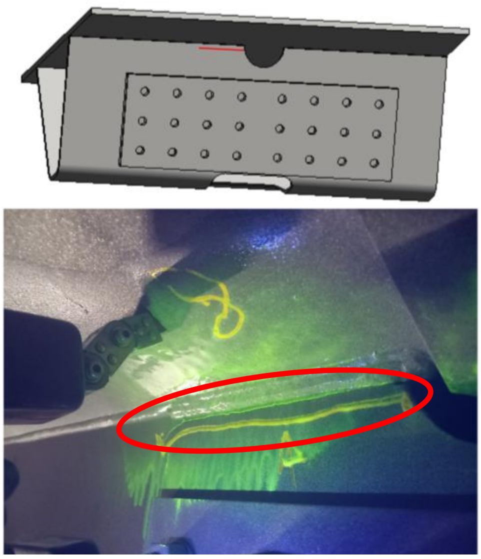

Figure 16: cracks at u-rib bolted joints

7 Reinforcements

In the following we present part of the reinforcements installed on the viaduct. Although obviously exploited also to ensure static stability, the reinforcements presented hereafter are limited to those required by the fatigue analysis conducted according to the procedure described in paragraph 4.2. Other reinforcements, that result from ULS verifications, are not discussed here. From a fatigue perspective, reinforcements represent additional details, for which future damage, D_2, must be < 1.

7.1 HPC slab

A 90mm-thick High-Performance Concrete (HPC) slab is laid over the original steel deck-plate (Figure 17). The purpose is to provide extra stiffness to the deck-plate and to diffuse axles loads to mitigate local damage. This probably represents the most effective reinforcement put in place during the renovation. Based on the experience of Netherlands bridges, the presence of a HPC slab even allows to accept cracks in the deck-plate up to 500 mm (cf. 6.1).

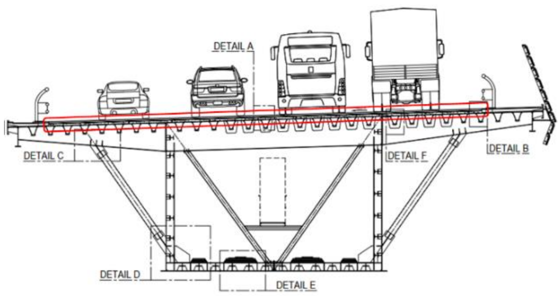

Figure 17: HPC slab

7.2 Patches

To avoid cracks developing from the cope-holes (cf. 6.2), double-side patches, of 15mm thickness each, have been installed around the u-ribs at the crossbeam intersection. Two kinds of patches are present: large patches, needed at the extremities of the central crossbeam, where the shear effort is the largest, and small patches, for other locations (cf. Figure 18). The shape of the patches has been optimized to avoid hard points in the deck-plate and not to induce too high stresses in the new patch-to-u-rib welds, as well as to allow future inspectability. Some patches are also needed for ULS verifications (cf. Figure 18).

Figure 18: Patches

7.3 Braces

Braces are installed to support the cantilever and relieve the welds connecting the cantilever to the box girder web (Figure 19). Braces are made of slitted CHS tubes welded to a gusset plate. Fatigue calculations highlighted the need to perform a very local full penetration weld at end of the tube slit, as this greatly improves fatigue performance compared to a classical filled weld.

Figure 19: Braces

7.4 U-ribs joints

U-rib bolted joints are replaced by welded connections. The original bolted connection is cut out and a new piece of u-rib is inserted (Figure 20).

Figure 20: u-rib welded joints

7.5 Weld reinforcement/replacement

Despite the addition of braces, the welds between the cantilever and the box girder web need to be reinforced (Figure 21). This is related to scenario 7b (4 traffic lanes), with trucks mainly driving on the cantilever. However, since root cracks in fillet welds are not inspectable, welds for which past damage $D_1 > 1$, cannot just be reinforced but must be redone entirely, as it is impossible to estimate their current state of damage.

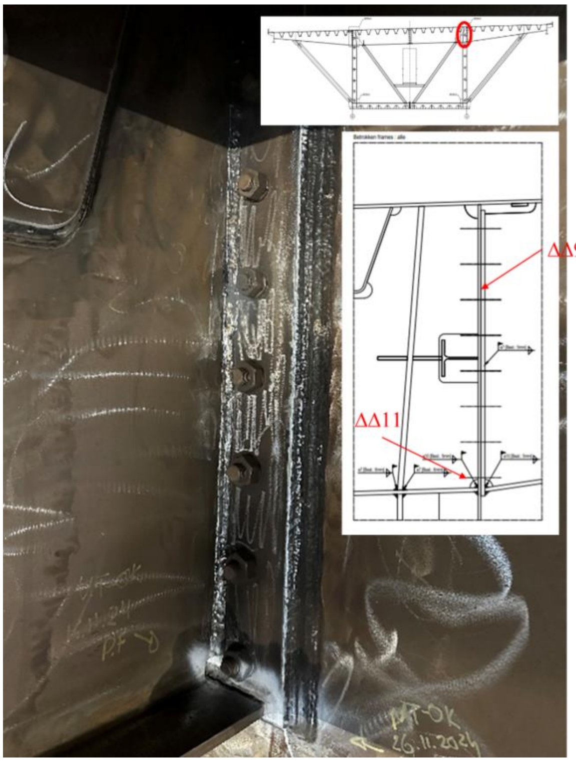

Figure 21: welds reinforcement/replacement

7.6 Weld toe treatments

Not only root cracks can be problematic in existing welds but also toe cracks. For that, surface treatments, such as High-frequency-mechanical-impact (HFMI), are applied on existing welds between the crossbeam and the u-ribs (cf. Figure 22). This procedure has two purposes, according to [5]:

- Remove very small cracks (depth < 2.25mm), undetectable with MPI (minimum detectable depth = 2mm);
- enhance fatigue performance by inducing a local compressive stress state.

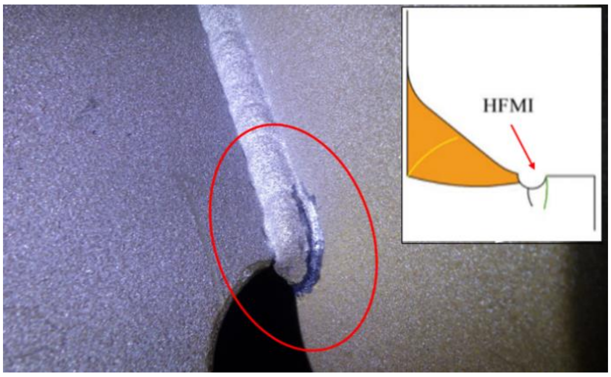

Figure 22: HFMI treatment at crossbeam-u-rib joint

8 Conclusions

The fatigue assessment and retrofitting of the Vilvoorde viaduct through a hybrid strategy combining Eurocode S-N curves, fracture mechanics and inspections, has been presented. The proposed approach allowed to consider both past and future damage, while limiting the areas to be inspected and/or repaired. The reinforcements installed, such as a high-performance concrete slab, u-rib patches, and braces, have greatly improved the fatigue performance of the viaduct. In conclusion, while the classical use of Eurocode SN curves [2] would have led to the replacement of the orthotropic slab and the upper part of the crossbeam, or even the entire box girder, this study allowed, by means of some reinforcements, to extend the service life of the viaduct until 2078, while accommodating up to four traffic lanes (two of which are 'heavy' lanes) on each viaduct.

9 References

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