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Fatigue reinforcement during repainting for two motorway bridges

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Abstract

A large number of metallic structures have already reached or even exceeded the limit of their initial design fatigue life. All this paper reports and investigates about the various works carried out in order to extend the fatigue life of existing motorway bridges located in the north of France.

The studied bridge is composed with two independent decks of a French motorway carrying heavy traffics for 40 years. The slabs of both bridge decks are very fine: steel-concrete composite slabs associating 8 mm steel plate and 100 mm concrete layer according to the original design of Charles Brignon, who was also a pioneer in France of modern fatigue design for bridges, using rounded gussets to attach transverse beams, which reduces the stress concentration factor at the crossing point of the flanges.
The structures of the bridges were recently strengthened to extend their fatigue resistance by three different methods which are:

- Addition of a continuous welded steel plate inclined outside of the edge girders to increase the safety at the Ultimate Limit State (ULS) and the robustness in fatigue of the bottom steel flanges. The inclined additional plate supported by the webs and the free bottom flanges
- Post weld treatment with Tungsten Inert Gas (TIG) dressing of the welded cross-beams connection gussets were performed.
- Bonding of CFRP carbon fibres on the bottom steel flanges of the main beams at the extremities of the welded existing cover-plates. These fibres need a high elastic modulus of 400 GPa.

The interactions between both fatigue and anticorrosion life extension works are discussed. An innovative option of mixed complete-partial sandblasting methods was proved more economic than the usual complete sandblasting

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method and performed. The initial conception with two independent decks carrying the highway showed itself particularly relevant for the implementation of heavy repairs. The traffic deviation on the other deck allowed to work on each deck without stopping the traffic.

Keywords: Road bridges; Fatigue design; TIG dressing; HFMI treatment; Carbon plate sticking; Corrosion; Blasting

1. Introduction

These works dedicated to fatigue life extension had to be actually carried at the occasion of the bridge repaint. Both works were for instance using the same scaffolding. These works have strongly mutually affected each other: the bridge repaint works needed to be planified and oriented in order to make also the fatigue life extension works possible. The improvement techniques and carbon sticking had to take place during the repainting works. As well for fatigue life extension works and for repainting works, different experimental techniques were tested. We hope that the discussions about those interactions between both fatigue and anticorrosion life extension works will be useful for future similar cases.

Bridges represent an important heritage of the society. Fatigue represent one of the major aging factors that affects their durability with the risk of collapse due to cracks propagation. The possibility to improve these structures to extend their fatigue life represent an attractive solution. In this paper a study of the reinforcement techniques used to improve the durability of Dancourt bridge as well as the opportunity of some other improvement methods as High Frequency Mechanical Impact (HFMI) are carried out.

2. Characteristics of the bridges and justification of the chosen design

The bridges of Dancourt, on the commune of Donchéry in the local French authority of the Ardennes, carry A34. This highway of the national network connects Charleville and Sedan. Circulation on A34, reached 30000 vehicles per day including 11 % of heavy trucks. The bridges cross the river Meuse in the west of Sedan. The refurbishment work presented are in detail described in **[1]**.

Two independent bridges constitute the highway platform. This conception ensures a great robustness for the highway connection itself. The choice of independent bridges appeared particularly relevant. Indeed, today for maintenance and allowed much more easily the implementation of heavy repairs, it was possible to work successively by alternation on each bridge while cutting it and reporting all the traffic on the other bridge.

These bridges were built in 1971/1972. Each one of them is carried by 5 steel beams ensuring to each deck an important intrinsic redundancy. The three spans successively have 37m - 66m – 37m of length.

The transversal distance between the five beams is 2,50 metres. Ends of the bridges are not skew. There is no geometric curvature in the plan and the longitudinal profile has a single slope. The steel used for the metal frame was S355, called A52 nuance during the construction time. For the transverse beams, steel was S235, called A42 during the seventies (construction time).

Figure 1 : ***Sight of the beams of the two independent bridges supporting the highway***

Moreover, these bridges profit from an excellent initial design due to Charles Brignon, who was in charge for bridges design in the Ardennes at the time of the bridges construction:

1) The bridges present already monolithic anti-fatigue gussets at the connection between transverse beams and main beams even if it was not demanded by the fatigue design rules when the bridge was designed.
2) Webs of the longitudinal beams do not present longitudinal stiffeners because their ends constitute weak points for fatigue. Today Eurocodes also tend to privilege this type of design.

3) The deck is a steel-concrete composite deck, which would still be regarded today as innovating, even if the first steel-concrete composite decks were built at the beginning of the 20th century for bridges crossing canal. The bridge in Figure 3 crosses the Bray canal parallel with the river Seine at Bazoche and it is always in good condition. It was built around 1900 and made of puddle iron. The bottom plate has a cylindrical geometry.

Around 1934, F. Leonhardt studied the steel concrete connection and systematized the steel-concrete composite decks to facilitate the construction of highway overpasses. J.R. Robinson used steel-concrete composite decks for large suspension bridges in Tancarville and Bordeaux.

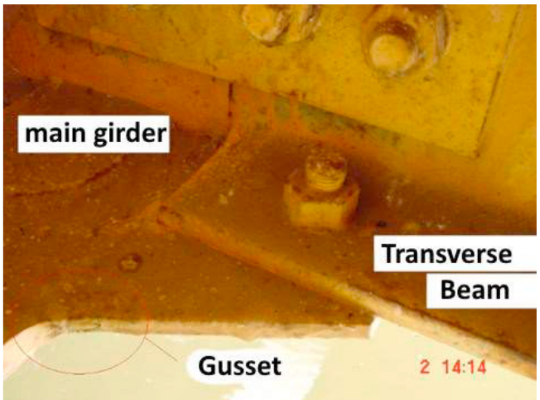

Figure 2 : ***Anti-fatigue rounded gussets (built in 1971 – 1972)***

Figure 3 : ***steel-concrete composite deck (built around 1900)***

For the steel-concrete composite decks of the bridge of Tancarville, Nicolas Esquillan has actually used studs for connection. But the high number of connectors was expensive. The thinness of bottom plate and the absence of stiffening constituted difficulties for the implementation on building sites. Many designers gived up the solution of the steel-concrete composite decks. The solution presented in **[2]** with stripes of CL-connection dowels may be a solution for the future to insure rigidity and connection at the same time.

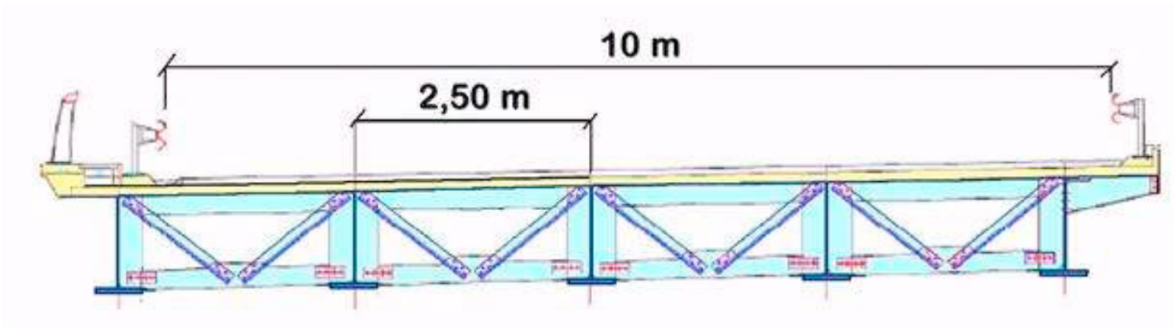

Figure 4 : ***Transverse section***

The steel-concrete composite deck that was designed by Ch. Brignon for the bridges at Dancourt consists of a uniform 100 millimetres thickness layer of reinforced concrete, connected to a metal bottom plate of 8 millimetres thickness. This plate is used both as formwork during construction and as shell in tension to carry traffic loads during service. Today the thickness of the steel bottom plate would be increased up to 12 or 15 millimetres in order to facilitate the assembly on building site.

On the other hand, steel technology available at the time of construction did not place plates with the sufficient thickness at the disposal of the bridge engineers. For this reason, cover plates (see Figure 5) were disposed on the outside of the beam flanges. Such cover plates would be avoided today.

With a low slope less than 1 / 3 of the front weld as on the Figure 5 would be favourable for the category of detail but the corresponding crack to fear is at root and can remain hidden a long time to inspection before being detected.

The slope of the front weld at Dancourt is near 1 / 2 and in this case, the crack that takes place likely in the front weld is at the weld toe. The recent results about cover plates as well as the crack location (at weld toe and at weld root) are available in [3].

The end-points of the cover plates were considered as weak points with a category of detail of 56 MPa.

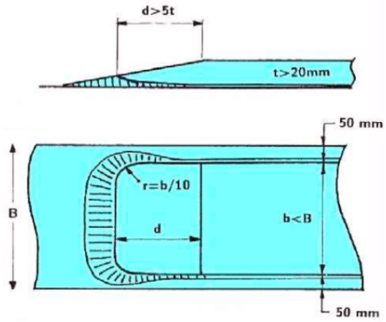

Figure 5 : ***Cover plates***

3. Assessment of the state of the bridges

The fatigue strengths given in the Eurocode 3, Part 9-1 apply to structures operating under normal atmospheric conditions and with sufficient corrosion protection and regular maintenance. The degradations observed on anti-corrosive protection were very unequal. They were much more marked on the outside of the lateral beams than on the intermediate beams (see Figure 6). That's why a special treatment of the outside of lateral beams was carried out.

Outside of the lateral beam ***Intermediate beams and bottom flange of composite deck***

Figure 6: ***Disparity of degradations of protection against corrosion***

3.1. Special investigations of the steel-concrete composite slab (October 2008)

Cerema (ex-Sétra) recommended complementary investigations to check the integrity of the waterproofing. It was checked that no water was retained between the reinforced concrete of the composite slab and its bottom steel plate. As a matter of fact, severe degradations can occur with freezing if water penetrates into the concrete and remains retained by the bottom plate.

The examination using the radar led to conclude that the original old waterproofing (see figure 7), had preserved its effectiveness everywhere. In addition of that, some drillings of control through the bottom plate revealed neither degradation, nor reduction in thickness dimension. The thickness of the bottom plate of the composite deck was even accurately measured to 8,6 millimetres, instead of the theoretical value of 8 millimetres that was expected according to the original design.

In addition, permanent mining devices that were still required by the military authorities in 1970 were actually also very bad fatigue details and could be removed with the agreement of the competent authorities.

Figure 7 : ***Radar examination of the composite slab and its waterproofing***

3.2. Recalculations carried out in 2008 revealing insufficiencies in original dimensioning

Calculations drawn up in 2008 by S. Neiers revealed insufficiencies of the initial dimensioning of the bridges with respect to the ultimate limiting states regarding the moments of resistance on piles. Effects of fatigue were also underestimated at the time of the design, even with reduced combinations of the Eurocode loads. Theoretical insufficiencies of the dimensioning of certain web thicknesses according to the theory of the linear elastic buckling were also detected.

4. Principal methods to retrofit the bridge

4.1. Sloping plates: Principle and advantages

Cerema (ex-Sétra) proposed the following reinforcements:

- It was decided to bond in a second step carbon-fibre plates with glue at the ends of the cover plates.

  It was also decided to add a continuous reinforcing plate over the entire length of the bridge

Outside continuous longitudinal sloping plates along all the bridge over the bottom flange of the edge beams was proposed and adopted (see figure 8). This way to add a new plate on site is easier to implement than a cover plate that has to be welded with "overhead" welds under the bottom flange. This continuous sheet does not introduce new mechanical discontinuity i.e. new long-term risks of fatigue cracks.

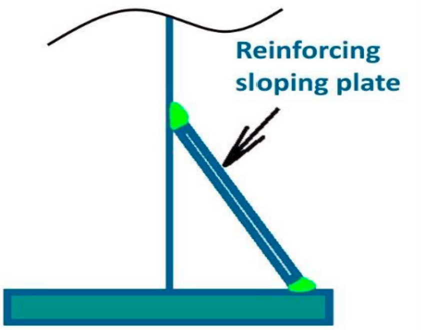

Figure 8: ***Reinforcement by sloping plates.***

Such sloping plates were already adopted for a new bridge of the ring of Strasbourg. It has been on an experimental basis carried out without problem in the case of this bridge even with an important geometric curvature in plan. The sloping plates are welded at their high point with the web and at their low point with the bottom flange. Other example using sloping plates are presented in [2] and [12].
On the other hand, the implementation of such sloping plates on an existing bridge is an innovation.

These sloping plates present the following advantages:

- ✓ **safety** by prohibition of any pedestrian advance on the bottom flange,
- ✓ **corrosion protection** especially at a location where the paintings were seriously degraded, by avoiding the water stagnations, and avoiding to provide nest places for birds on the bottom flanges.
- ✓ **rigidification regarding torsion** of the bottom flanges, without any risk of distortion which facilitates the justification of the bottom flange regarding elastic instability, in particular in the case of a bridge with a curvature in plan.
- ✓ **strong stiffening of the web** regarding buckling,
- ✓ **improvement of the transfer of shear lag** between the web and the bottom flange. The benefice of this addition is important at the ends of the existing cover plates, as well at each point where the thickness of the bottom flange changes. Aim is in both cases to reduce of the stress concentrations.
- ✓ **reinforcement of the bottom flange regarding the impact of oversized vehicles.** It was not the case for the Dancourt bridges but it was determinant and efficient for the bridges in Strasbourg presented in Figure 9.

*Figure 9 :****Bridges of the ring of Strasbourg at the crossing of the street ''des Bouchers''***

4.2. Implementation of the sloping plates

The additional sloping plates have a thickness ranging between 20 and 30 millimetres according to their location at pier or in the centre of the spans. The company chose to realize it by segments of four metres length. The different segments had to be initially linked with each other, before being welded by two longitudinal weld to the existing beam. This phasing limits the introduced residual stresses. The implementation could have been compromised by the insufficient preparation of the company. The two chamfers were opened in the factory to a total opening of 90 degrees instead of 45 degrees for the addition of the two angles. In addition, the old vertical welds between the existing web segments had to be locally grounded down to allow a correct implantation of the additional sloping plates along the webs.

After correction of these initial errors, the other geometrical imperfections of the beams were remaining within the acceptable limits to allow the implementation of the additional sloping plates and their welding (see figure 10).

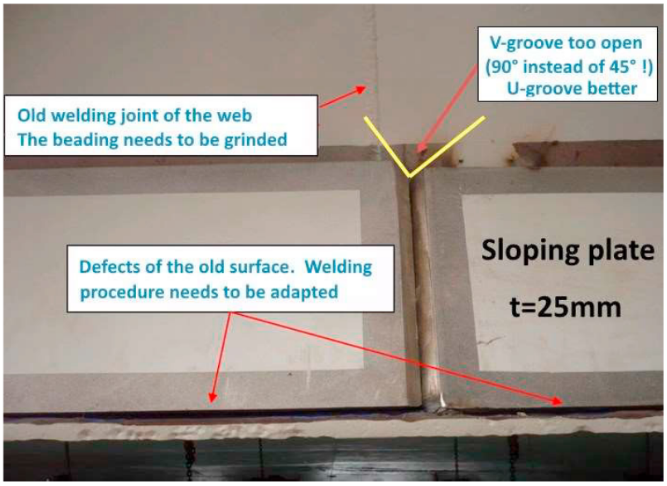

Figure 10 : ***Additional plates before welding***

The welding of the additional sloping plates revealed some pre-existing imperfections of the basic bottom flanges by causing some cracks in a horizontal plane crossing these flanges. The old plates of the seventies had been elaborated before the implementation in the forging mills with continuous casting. Some delamination defects of origin due to impurities were opened by the welding of the additional sloping plates.

The Cerema (ex Laboratory of the Ponts et Chaussées of Nancy) had to investigate the ends of cover plates: visual and by ultrasounds (US). In addition, a more complete inspection of the flanges was also carried out. The investigations by ultrasounds showed that the depth of the delamination cracks does not exceed 40 millimetres deep. These investigations could also determine the localization of the delamination cracks reigning over lengths between one metre and four metres for longest. The defects revealed at this occasion are in fact without danger for the stability of the bridge and pre-existed the operations of reinforcement by additional sloping plates.

It was thus decided to simply weld shot the delamination cracks revealed by welding the additional sloping plates. A fillet weld of minimal thickness (=< 5mm) was deposited in the longitudinal direction along the cracks on the edge of the bottom flanges. These fillet welds treat each zone detected by US investigation with a margin excess-length of 300 mm at each end of the cracks (Figure 11). In addition to its mechanical role, this closing of the interstice stops corrosion there by preventing any renewal of oxygen.

Aspect just after detection (Photo J.Y. Joineau) ***Aspect after treatment***

Figure 11 : ***Treatment of the inter-lamellate longitudinal cracks***

4.3. Complementary H.F.M.I. treatment possible but not realized

It was decided in 2008 to avoid welding work at the ends of the cover plates to extend their lengths. The new cover plates extensions welded under the bottom flange of the lateral beams under difficult conditions of "overhead" welds. In addition, these operations would have introduced, after cooling of the weldings, important residual stresses of traction prejudicial to the durability of the assemblage.

A - ***View of the ends of the cover plates with the carbon fibers stripes*** **B** - ***SLS Stress concentration at cover plates ends before reinforcements***

Figure 12 : ***Geometry of the cover plates ends of the Dancourt bridges***

Figure 12 shows the geometry of these cover plates and the results of a recent first calculation of the stress concentration (von Mises criterium). The results of Figure 12-B are only qualitative then the effective notch stress method is required to evaluate the local stresses. In addition, this method was not available at Setra in 2008 when a decision had to be taken regarding the Dancourt bridges.

During the reinforcement time the TIG dressing was very known and used in several other bridges. It was selected to treat the existing cracks for crack lengths less than one millimetre. Mechanically the TIG removes the cracked zone and improves the geometry which leads to reduce in the stress concentration at the weld toe. This improvement leads to retardation of the crack inhiation phase [16]. But TIG dressing cannot be recommended as the best treatment at the centre of the cover plates ends (see figure 12-B) since Fisher et al. [19] noted that improvement of cover plate by TIG dressing at the weld toe is sufficient in a few instances to force the crack to originate from the weld root. In addition, no cracks were detected at Dancourt at cover-plate ends that needed to be repaired by TIG dressing at weld toe.

Today other new techniques to treat the fatigued welded zone have also emerged. One of them is High-Frequency Mechanical Impact (HFMI). HFMI is a recent improvement method to treat the surface of welded zone which is probably one of the most effective for treating welded assemblies. In HFMI treatment the welded zone is bombarded by small hard shots leads to plastic deformation of thin surface zone and compressive residual stress. This compressive residual stress (RS) leads to improve the fatigue proprieties of the material by increasing the surface resistance to crack initiation and the retardation of crack propagation [17] and [18]. With this treatment that can be recommended both improvement in the geometry (to reduce the stress concentration factor) and introduction of a beneficial residual stress at the surface are noted.

These HFMI treatments will make possible to increase the fatigue life of such cover plates ends by mainly increasing the initiation period of fatigue cracks.

4.4. Complementary reinforcement with carbon fibers plates CFRP actually realized

The ends of the cover plates were actually reinforced with carbon fibre plates in 2008. The implementation of the carbon fibre plates supposed that the companies answer by proposing offers and materials adapted to be bonded by glue on steel. The module of usual carbon fibres used for the concrete structures is indeed 160 GPa only. Such fibres cannot be used for being bonded by glue on steel because they would have taken only one small part of the service loads. Their fatigue-stress reducing impact at the ends of the cover plates would have been too small.

The company proposed **Sika-Carbodur** plates with a more important density of fibres. Their elastic module in traction is of **400 GPa** and this rigidity is higher than that of the laminated steel whose theoretical elastic module is 210 GPa. It should be noted that the measurement of this module for the steel of rolled plates is difficult and often reveals a discrepancy between 200 and 230 GPa.

The reinforcements by CFRP were carried out under the slow way of the trucks on the right of each bridge, i.e. on the bottom flange of the edge beam (Beam n°1) and on the close neighbour beam (Beam n°2). Thanks to the lightness of the CFRP, they can be implemented as well above or below the bottom flange. They can also be implemented on one face or on the other face of the web, according to the presence or not of an intermediate stiffener in the zone to be reinforced.

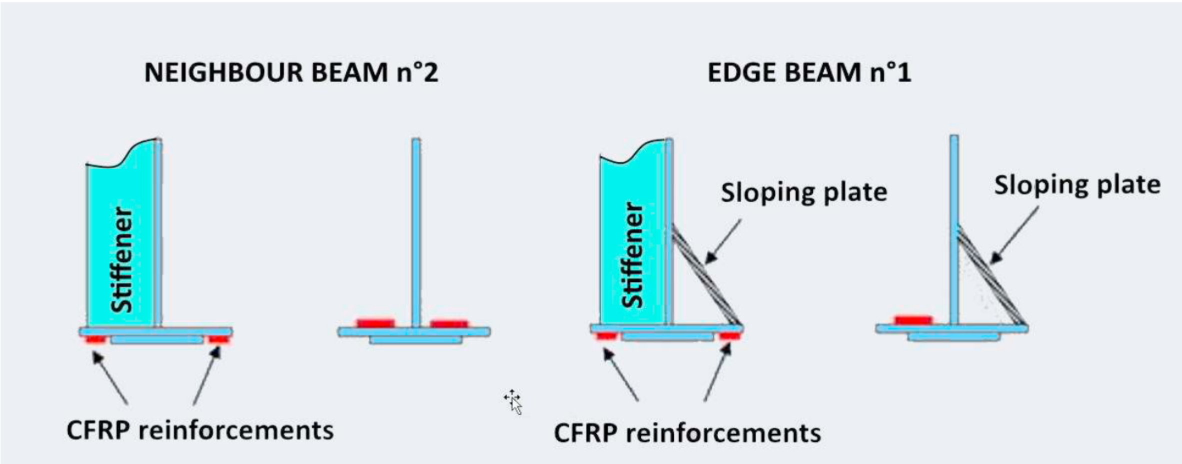

Figure 13: ***CFRP reinforcement plates of carbon fibres with 400 GPa of elastic modulus***

A calculation with the finite elements led at Cerema by Fabien Rizard using Code-Aster showed that the optimal disposition of the three CFRP layers must preferably respect the shifts of figure 14.

The 80 mm length of the step width makes it possible to reduce the stresses of approximately twenty percent.

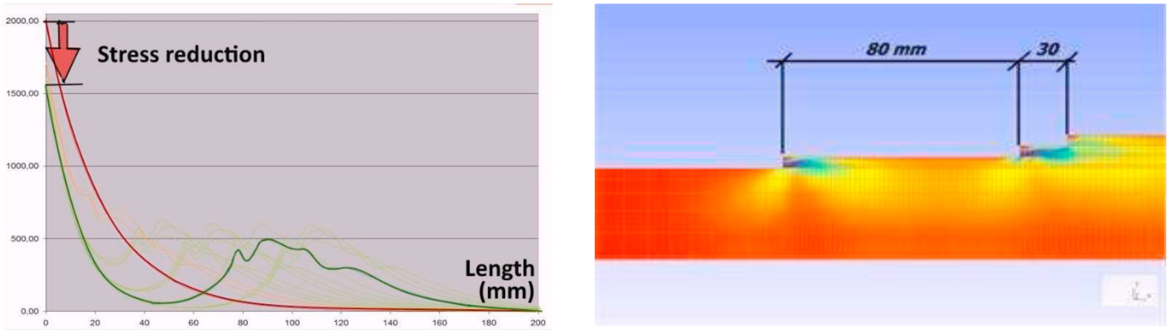

*Figure 14 : **calculation of the optimal geometry of the CFRP layers using Code-Aster***

Figure 15 : ***Implementation of the carbon plates***

The reinforcements by carbon fibers with the optimal geometry was only possible for the second bridge. The Figure 15 shows the implementation for the first bridge. This implementation was controlled by the Laboratories of Ponts et Chaussées of Strasbourg and Autun.

4.5. Post weld treatment of the gussets by the technique of TIG dressing

The existing monolithic gussets present a single general plate of connection with a trapezoidal form. The bridges of Dancourt built in 1971 were already equipped with rounded radius transition to plate now envisaged in Eurocodes. But in the original design, the radius was only of 45 mm instead of the 150 mm required today by the norm. Aim of the T.I.G. treatment was to regularize the geometry of the gussets by increasing their radius locally. TIG treatment was applied in 2011-2012 for the reinforcement of the bridges of Dancourt. The remelting of welds in this case (4mm of treated cracks) was supposed to resets the cycle clock to zero which is not the case in other deep cracks.
A calculation of the zones of stronger probability of crack initiation, was realized by Cerema using Code_Aster (Open source software developed by the French company for electricity supply EDF, R & D). The results made possible to specify the importance of the zone to be treated by TIG dressing. This zone is very limited and corresponds to the zone with red color of figure **16**.

The principles of 3D-volume modeling presented here were developed at Cerema (ex Sétra) for the European research program Precobeam. These principles made it possible to find by calculation the classes of fatigue of the Eurocodes based on experimental results as presented in **[8], [9], [10] and [11].** Evaluation regarding fatigue for various types of hangers used for tied arch bridges as those described in **[12]** were achieved in 2018 with the same method to evaluate existing constructions **[13].**

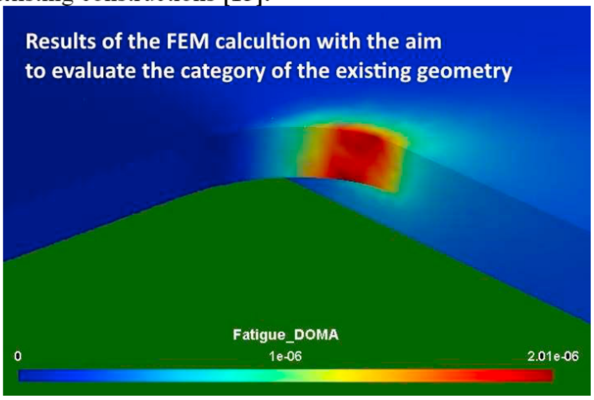

Figure 16: ***Zone with fatigue crack risks, and result of the T.I.G. dressing of this zone***

In the zones of the welding, the size on the sides of the 3D-volume mesh is of 4mm, it is entirely carried out by tetrahedrons not very different of regular tetrahedrons in the zone of the hot spots. This grid is in conformity with the recommendations of December 2008 of IIS-IIW (International Institute of Welding) **[14].** In each mesh, the stresses are calculated by Code_Aster at the Gauss point. The extrapolation of the recommendations of December 2008 of the IIS-IIW is carried out industrially for all the tetrahedrons using the GMSH smoothing function of the results.

According to the previous procedure, the FE model made it possible to evaluate the class of fatigue before TIG dressing: for a radius of 45 mm the class of fatigue is 60 MPa when it would be 80 MPa according to the Eurocodes **[15]** for a radius of 150 mm.

It was not necessary to treat all the transverse attachments, but only those which present important irregularities of geometry or were located in a sensible section regarding the fatigue calculations of the bridge.

5. Repair of anti-corrosion protection

The red lead anti-corrosion protection of the steel beams of the two bridges was almost 40 years old at the time of repair work. The situation could thus have justified a complete changing of the protection to eliminate all lead from the bridges.

The anti-corrosive protection of original to red lead is indeed difficult to renovate after blasting. It requires to take precautions as well with respect to the **hygiene and safety on the building site** as for the **environmental protection,** in order to **avoid any intoxication of workers and any pollution of the River Meuse.**

Because of the very strong disparity of degradations among the old paintings highlighted by the photographs of figure 4, the adopted solution consists in complete blasting and renovating only the very degraded zones (edge beam webs, specific bolted connection, zones on abutments, …). This is very economic and limits the risks of dissemination of lead and other toxic chemical pollutants on the building site. It is true on the other hand that extended zones that had remained in good condition, will stay protected by lead.

5.1. Organization of the call for tender for repainting work

The class of environment or category of corrosivity is the C4 class according to the norm ISO 12944-5. The tender documents drew the attention of the companies at the unusual complexity of the building site, which included:

- ✓ Complete blasting of some described zones: edge beam webs, specific bolted connection, zones on abutments,…
- ✓ Careful sweep blasting of the zones where old lead protection will remain,
- ✓ Investigations at different working stages (Dye penetrant inspections, US inspections,) to search for possible fatigue cracks,
- ✓ Reinforcements of the edge beams with additional sloping plates, and bonded CFRP,
- ✓ Repainting with a new system including a primer with zinc in the completely blasted zones.

The documents of the call for tender specified in a plan the zones to be sweep blasted and the zones to be blasted. The analysis of the offers took account of the organization adopted to ensure the quality and the durability of the work of maintenance by coordinating the various subcontractors that were involved in a detailed chronogram.

Figure 17 shows the steps for the re-painting work. The quality of the technical proposals of the companies (containment, ventilation, organization of the dressing rooms for the workers, measurements of control) belonged to the criteria of judgment of the offers. Abrasive and UHP-water were the two possible techniques for sweep blasting.

Figure 17 : ***Intermediate transverse beam at different blasting levels***

5.2. Implementation of anti-corrosive protection

Blasting with abrasive can pose acute problems of hygiene and safety since the building site is hermetically wrapped to avoid any rejection in the environment. It was only possible because of rigorous dispositions taken to allow the protection of the operators and the controllers:

- ✓ well dimensioned systems of aspiration and ventilation are an essential point to answer the hygiene and safety priority concern to avoid any intoxication with lead,
- ✓ wearing of masks on building site,
- ✓ passages by a sluice of decontamination and a dressing room at the occasion of all types of pause, for toilets or meal in particular. It is easier to write down on the contract-paper this last point, than to ensure its respect in practice.

The final painting layer has to be compatible with the various primary systems containing lead and or zinc.

It had been initially estimated that one would treat 20% of surface by complete blasting and 80% of surface by sweep blasting. The distribution actually implemented is ultimately 30% and 70%.

Figure 18: ***Dressing room and masks for hygiene and safety***
Containment of the working site to protect the environment, and strong vacuum cleaners.

6. Conclusions

Important and innovative work was achieved to extend the fatigue life of the Bridge Dancourt:

- ✓ Welding of a **sloping plate outside the lateral beam** to increase safety at ULS and the robustness in fatigue,
- ✓ Application of techniques of post weld treatment **by TIG dressing** on the gussets,
- ✓ **Sticking of carbon fibres** plates on the bottom flange of the beams at the ends of the existing cover plates.

The fatigue strengths given in the Eurocode 3, Part 9-1 apply to structures operating under normal atmospheric conditions and **with sufficient corrosion protection and regular maintenance**. An innovating option mixing **sweep-blasting** and **blasting** was proved more economic for this maintenance than complete **blasting**:

- ✓ **complete blasting** (approximately 30% of surface): lateral outside beams plates and zones on abutments
- ✓ **sweep-blasting** (approximately 70% of surface): intermediate beams and under-face of the plates of the composite deck.

The initial choice of two independent bridges to carry the highway appeared particularly relevant for the implementation of heavy repairs. A deviation with the two directions of circulation on the same bridge, made it possible to work on each bridge **minimising the inconvenience the road users,** during each of the two estival periods of 2011 and 2012.

Throughout the working periods, information campaigns of the road users were carried out by various supports complementary to road indication with variable message sign bridges: **press pack** were developed and diffused, and press **press communiqués were released**, information telephone number and Internet site were maintained. The users thus could have reliable road information permanently in order to adapt their way, to take account of the new transit times, and to modify their behaviour when approaching the building site.

Acknowledgements

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