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Evaluation regarding fatigue for various type of hangers used for tied arch bridges.

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Abstract

The roads represent an important heritage owned by the French Ministry of Transports. Even more than corrosion, the fatigue is the principal aging process that affects the durability of steel bridges. Several examples illustrate in the article the importance of affecting a right consideration to the fatigue design of bridges.

Details that may appear as accessory to most of the usual bridge designers may be in fact of a crucial importance. It is in particular the case of the welded joints. The fillet joints are much more sensible to the fatigue stresses and should be avoided when there is a doubt, because their evaluation is very complicated. If the designer of a bridge uses such attachment, he has to verify the relevance regarding fatigue **before calling for tender** because the time for such studies is generally not available during the execution studies.

Calculation FEM techniques are used to evaluate the **stress concentration factor** that has to be taken into account for the fatigue design. For bridges, many fatigue details are classified in the Eurocode 3 (part 9) from tests. But some details that cannot be found in the Eurocode, and can however be studied and evaluated by computation. Several examples of tied arch bridges are presented. The article presents also an example where the fatigue class of the detail regarding longitudinal stresses is evaluated thanks to a FE-modelization according to the 2008 IIW Fatigue Recommendations at the location of the attachment of a hanger.

Crack in a similar fillet weld of the attachment of the stiffener of a box girder bridge is presented to illustrate that fatigue may really cause severe trouble.

Peer-review under responsibility of the scientific committee of the 7th International Conference on Fatigue Design.

Keywords : Road bridges; Fatigue design, hangers, Fillet welds, **stress concentration factor;**

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1. Experience regarding the design of tied arch bridges

The author has a long experience in the design of tied arch bridges. At Sétra he designed the Saint-Gilles Bridge over River Rhone [1] with cable-stays for a span of 120m (figure **1**) and the much smaller Roboul bridge of figure **2** for the East-Pyrenean local Authority in France [2].

Sétra was involved with the Roboul bridge in the conception of a pilot-project for a Mediterranean bridge allowing to cross a river in one span without intermediate piers to reduce the hydraulic impact, the risk of the soil to be washed away near the intermediate pier, and to preserve upstream area from catastrophic flooding [3]. For this purpose the Roboul bridge presents welded suspending rods which are elegant, rustic and economic. The economic pertinence of steel concrete tied arch bridges was proven at this occasion even for a small 40 meters span.

Sétra also participated to the conception of the bridge of Bédarieux for the French Hérault Local Authority in 2009. This bridge (figure **3**) shows that a span of 90 m with the same type of welded suspending rods as in for the Roboul-river crossing. If need be, it is of course also possible to embed this type of bridge on the abutment to remove the road joints and to constitute an integral bridge.

The Ko-We-Kara bridge for the New-Caledonian Southern Province is an other example of realization of this solution designed by Sétra for the structural as well as for the esthetical design [4].

All these bridges present a radial disposition of the hangers which simplifies and standardizes their connections. All upper connections to the arches are the same.

Figure 1 : Saint-Gilles bridge crossing the Rhône

Figure 2 : The Roboul bridge and his welded hangers designed for fatigue

Figure 3 : The Bédarieux 90m bridge

Figure 4 : The Ko-We-Kara bridge in New-Caledonia

2. Right constructional features usually required for hangers

For all bridges presented in part 1, the transmission of the forces is handled by a continuous plate and this plate is passing through a opening cut in the upper flange of the longitudinal beam regarding the lower anchorage. This plate is passing through the bottom flange of the box regarding the upper anchorage.

The figure 5 shows the lower anchorages of the Saint-Gilles bridge.

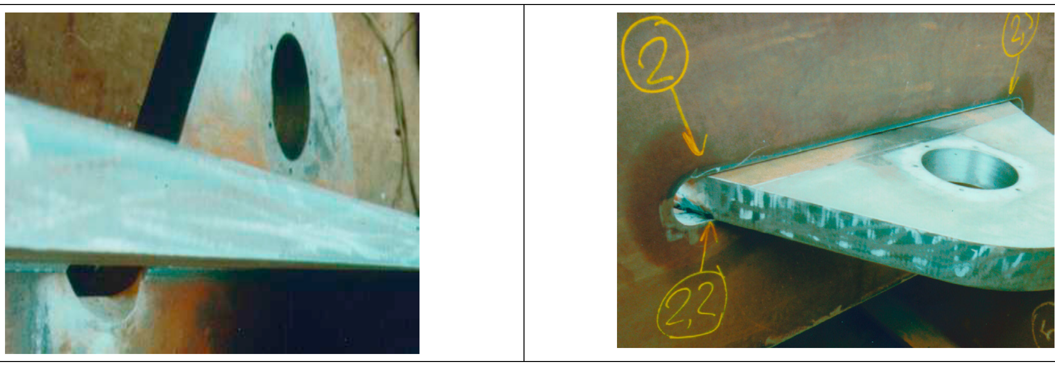


Figure 5 : lower anchorages of the Saint-Gilles bridge with fork sockets

For the Saint Gilles bridge the upper anchorages are designed for fix cylindrical sockets (figure **6**). The cable passes through the box into a tube and the forces are conducted to the arch through the diaphragm with fillet welds transmitting a force parallel to the fillet weld.

Figures **7** and **8** present details of the Ko-We-Kara bridge.

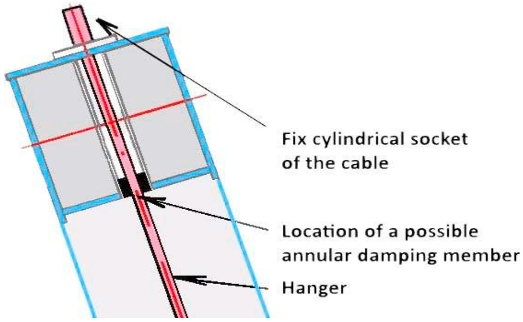


Figure 6 : Upper anchorages at Saint-Gilles

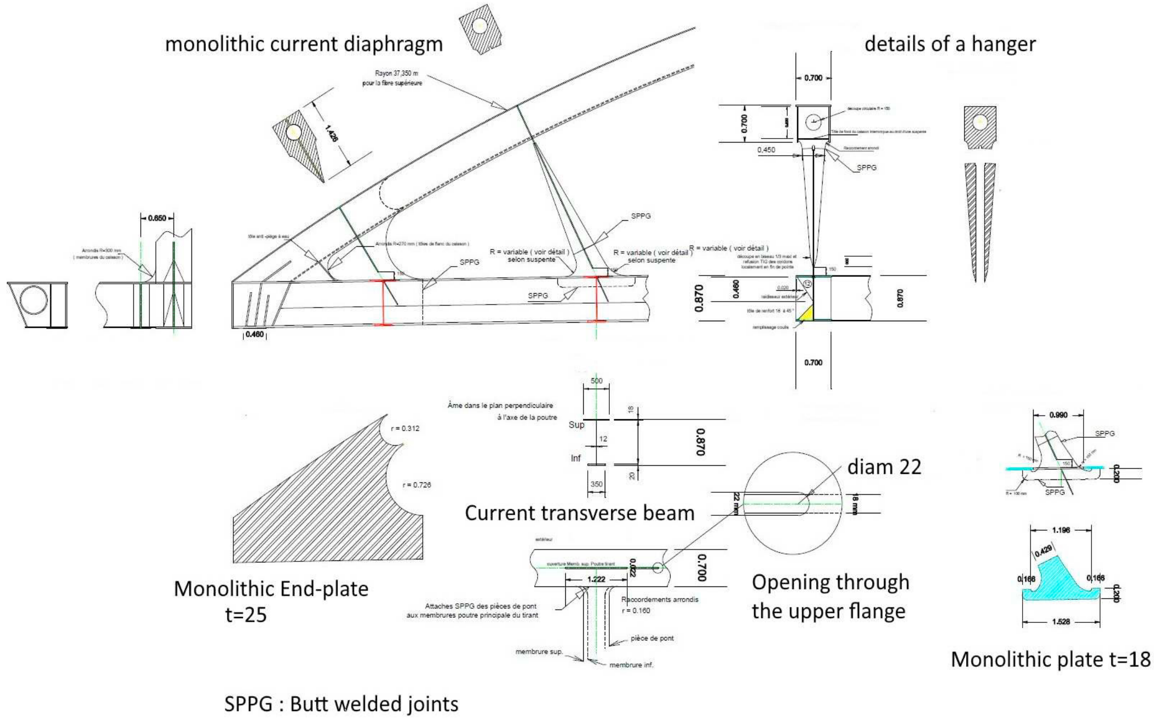


Figure 7 : Details of the Ko-We-Kara bridge before calling for tender

Figure 8 : lower anchorages of the Roboul bridge

The hangers were riveted or bolted for old bridges.

For all bridges more recently built in France before the publication of the Eurocodes in 2011, the welded double fillet welds on the left of Fig. 9. were avoided with the preference for an attachments using a continuous plate.

This plate is attached with fillet welds at the webs of the arch so that shear force is parallel to the fillet weld. Moreover, the bottom flange of the arch in compression is interrupted to insure the continuity of the diaphragm plate insuring the attachment of the hanger.

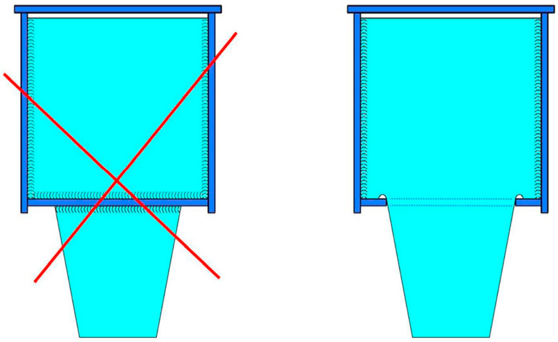


Figure 9 : Constructional features usually adopted for tied arch bridges

The two examples presented in the following parts of the present article will show the crucial importance of the right evaluation of the stress concentration factors in the application of the Eurocodes if one wants to deviate from the recommended features presented above.

3. Case of the upper anchorages of the hangers of a railway bridge

3.1. Presentation of the problem

The constructive detail of the connection arc-suspending rod is represented by figure 10. The force is introduced by a member with pin hole that should be of sufficient size to distribute the load from the area of the member with the pin hole into the member away from the pin.

This member (t = 30 mm) is assembled at the bottom of the box of the arc (t = 35 mm) by a T double fillet weld a=10 mm. At the interior side of the arc, the diaphragm is welded by an even weaker double fillet weld a=5mm.

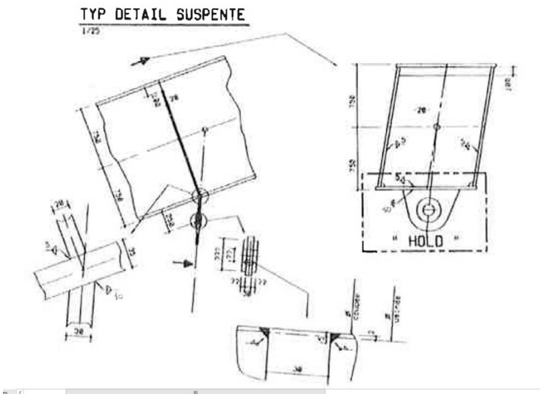


Figure 10 : Details of the concerned attachment

The fillet welds were designed assuming an uniform distribution of the stresses along the length of the fillet welds. But the fatigue stress is purely in the elastic field and no plastic redistribution is possible. In the case of such a pin member, the stress concentration factor has to be evaluated by a finite elements model because the effects of a concentrated loading and of the discontinuities of section can go in different directions as presented in figure **11**.

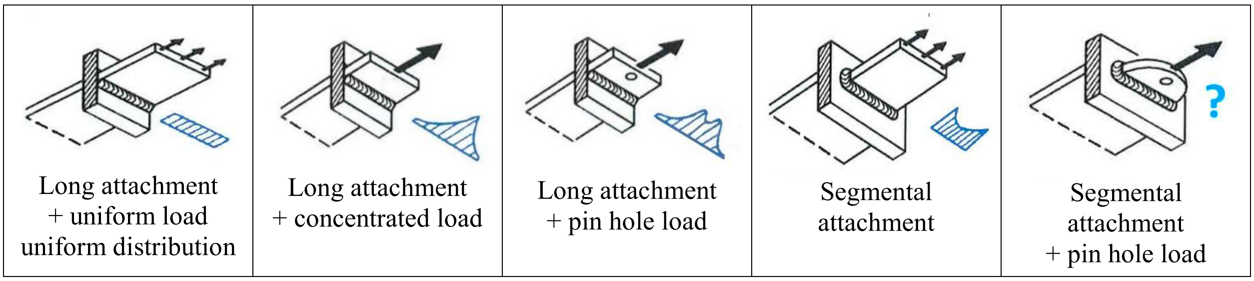


Figure 11 : Crucial importance of the right determination of the stress concentration factor

3.2. **Modelization**

The principles of 3D-volume modeling used 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] and [10].

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, Prof. Dr. A. Hobbacher and al.) [7].

In each mesh, the stresses are calculated by Code_Aster (Open Software of EdF R&D) at the points of Gauss. The extrapolation of the recommendations of December 2008 of the IIS-IIW is carried out industrially for all the tetrahedrons by the smoothing of the results by GMSH.

The adopted geometry makes it possible to model with sufficient precision the external fillet welds (a=10 mm) as seen on figure 12. The stresses in the structural plate at the toe of the fillet weld can be evaluated, as well as the stress at the root of the fillet weld.

As shown by figure **12**, the 30 mm thickness of the anchoring plate is divided for example in 7 segments of 4.28 mm.

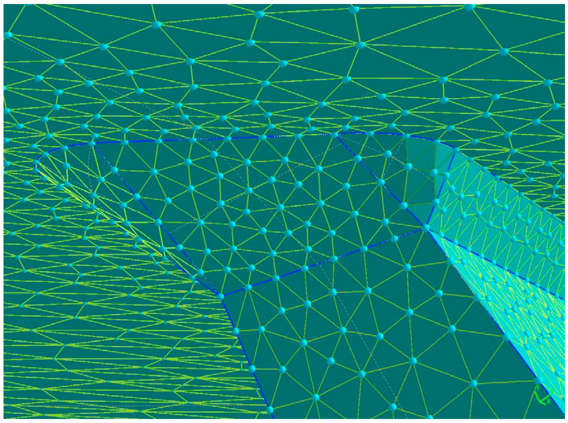


Figure 12 : Details of the mesh

3.3. **Results**

In a first qualitative approach, it is useful to have a look to the distribution of the von Mises criterion signed with the sign of the trace of the Cauchy stress tensor.

It is represented in figure 13. The load case is the ULS (Ultimate Limit State) load, but the calculation is purely elastic. However, the yielding stress is locally reached with a maximum of 450 MPa for ULS loads at the returning ends of the double T fillet welds.

As the fatigue load effect only remains in the elastic domain, the dominant principal stress of the Cauchy stress tensor is better adapted for the calculation of the stress concentration factor.

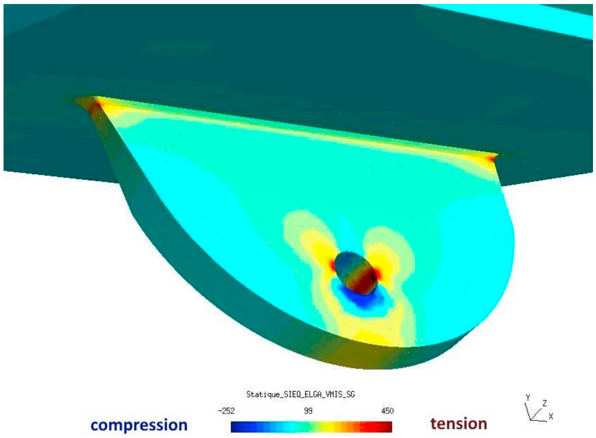


Figure 13 : von Mises criterion at ULS

A second model is needed to calculate the stress concentration factor, where the stresses are distributed uniformly in accordance with the assumptions of the designer and builder. The conditions of support are selected to obtain by symmetry an uniform distribution of the stresses as if the anchoring member had an infinite length.

Then the principal stresses are calculated under the same conditions for the two geometries. The von Mises maximum reaches for example 155 MPa.

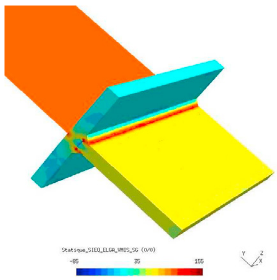
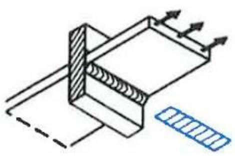
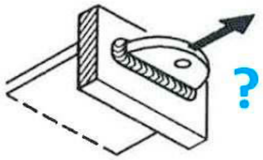


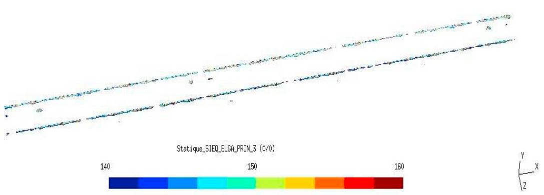
Figure 14 : von Mises stress at ULS with uniform stress distribution



Reference case : Uniform distribution

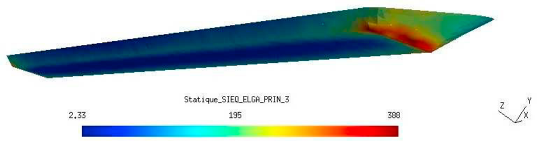


Real attachment case



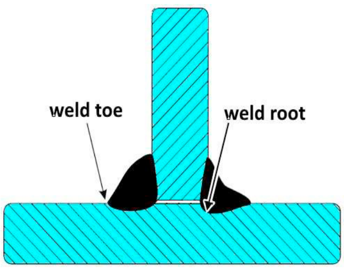
Dominant principal stress n°3 : selected average value

σ_3 = 144 MPa at weld root



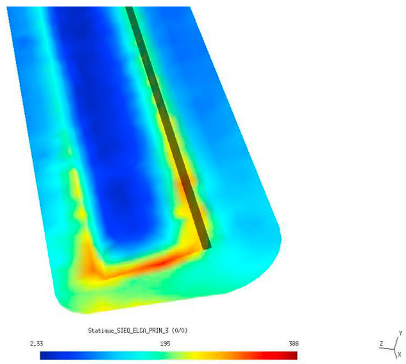
Maximum of **σ_3 = 388 MPa**

at weld toe on the turning places of the double fillet weld at the angle point



Stress concentration factors

f_{toe} = 2.69 **f_{root} = 2.43**



σ_3 = 350 MPa at weld root

Figure 15 : Table of the dominant principal stress n°3

The figure **15** presents the most important results. Cracks in the fillet welds of this size occur generally in the weld root, but the toes are here under severe stresses in the angles of the returning place too, so that the crack of Figure **24** is also possible. Usually The fatigue life is reduced from 100 years to about 15 years, value that can only be precisely evaluated with data concerning the traffic, and with a cumulative evaluations of the both effects of traffic and wind.

Moreover, all the suspending rods present the same design defect and not only the only one which is studied here. When the brittle fracture of a first suspending rod will occur, it is extremely likely that several close suspending rods also present fatigue crack initiation. When identical parts are subjected to very close stresses, they usually present similar fatigue cracks, especially when all the suspending rods present the same type of assembly, sensible and impossible to inspect. This domino effect corresponds to a negative correlation between resistances of the suspending rods which cancels the favourable effect of the redundancy i.e. the multiplicity of the suspending rods, according to the theory of the reliability of constructions **[6].** It is then appropriate from the point of view of the application of the standards, to change the Eurocode coefficient :

adopting γ_{Mf} = 1.35 rather than γ_{Mf} = 1.15 for a safe life in Table 3.1 of NF EN 1993-1-9.

Inside the box, the welds are also in trouble then the stresses are doubled (a=5 instead of a=10mm).

The effect of redistribution of plate of 35 mm will be evaluated later, but the reparation works will of course have to deal with both faces.

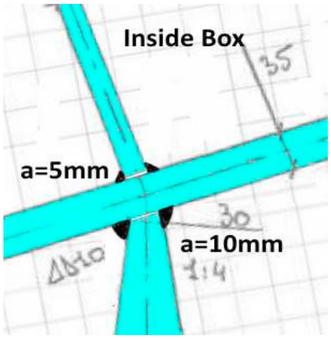
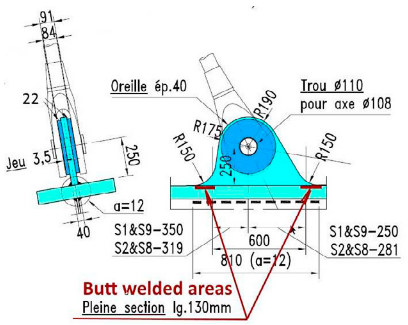


Figure 16 : Double T fillets welds inside and outside the box

4. Case of a lower anchorage of a road tied arch bridge

In the case of this second bridge, the pin-connection member is not in conformity with the recommendations presented in the part 2 of the present article, but is partially butt-welded over the upper flange of the bridge main girder. The total length of the attachment is 810 mm and two butt-welded area of 130mm were realized after the execution studies at each end of the attachment.



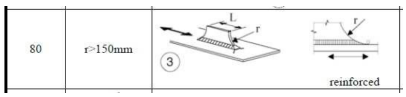
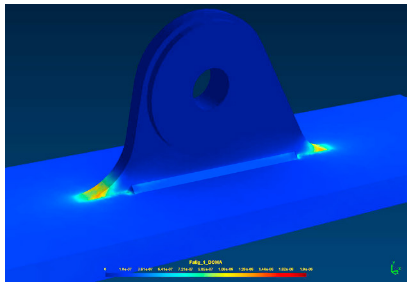


Table 8.4 of EN 1993-1-9 provides a class of **80 MPa** for the longitudinal upper-flange due to the weld attachment. This class is independent of the thickness of the attachment element.



FEM model of the attachment

Figure 17 : Attachment of the hanger on the upper flange

4.1. Evaluation of the class of the detail regarding longitudinal stresses

The fatigue strength of many bridge details under the effect of the longitudinal direct stresses caused by flexion compression or tension, are already classified in the EN 1993-1-9 on the basis of experimental results.

The Figure 18 shows most of the current bridge details and their fatigue class.

However, for details that are not classified, it is possible to evaluate the class according to the IIW method proposed by A. Hobbacher in [7] and already used by the Precobeam research project in [8] [9] [10] and [11] for the classification of the detail of the CL dowel connector.

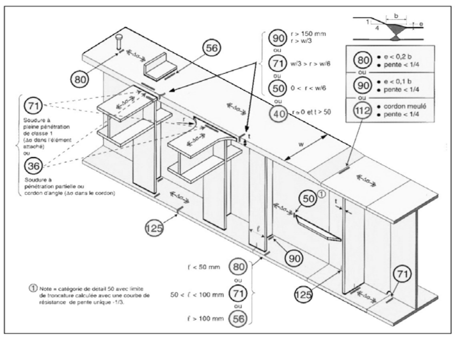


Fig. 18. Bridges fatigue strength details for direct stress ranges according to Eurocode 3 (EN 1993-1-9).

The intuition suggests that the impact of an attachment is growing in fact with the thickness (here 40 + 2*22 mm) of the attached element, and with the length (here 810mm) of this element. The result of the finite element analysis confirms this intuition and for the concerned bridge the right class of fatigue can be evaluated to 60 MPa instead of 80 MPa when the effects of the real geometry is taken into account.

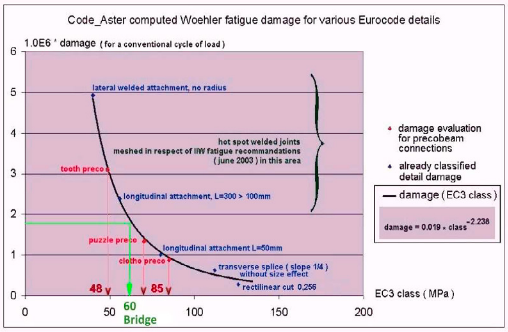
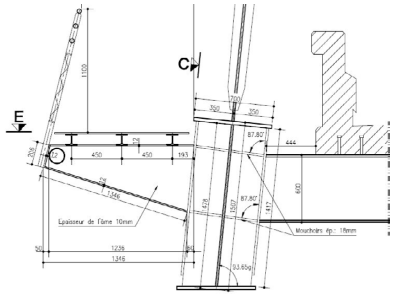


Fig. 19 : Evaluated class of fatigue according to the method developed by the author for the Precobeam research project.

4.2. Attachment of the cross beam under the hanger

More important problem with the conception of this bridge is at the attachment of the transverse cross beam at the location of the hanger. To evaluate this question a local model was built.

View of the realized attachment



Alternative better design for the Bédarieux bridge

Fig. 20 : Attachment of the cross beam at the location of the hanger

This local model is inserted in a global model of the bridge and the Eurocode fatigue load FLM3 is applied to the concrete deck as it is shown in figure 21. The result are presented in figure **22** without any ponderation with safety coefficients i.e. with $\gamma_{Mf} = \gamma_{Ff} = 1$.

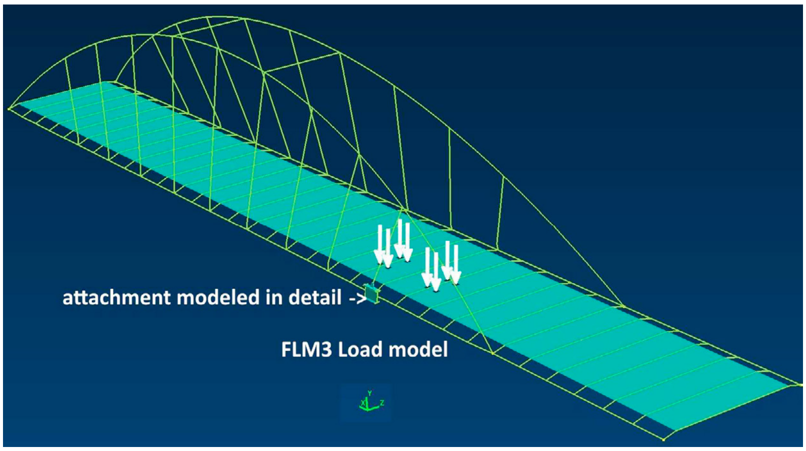


Fig. 21 : Global model and location of the FLM3 model of a fatigue truck

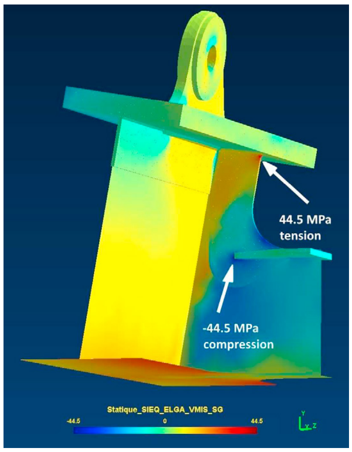


Fig. 22 : Signed von Mises

The weld fillets are in the 3D-model and the stresses presented in figure 22 are calculated at the hot spot at the weld toe in the supporting plate, i.e. the stiffener for the liaison between the stiffener and the upper flange of the main beam of the bridge. In a first qualitative approach, the figure 22 shows the distribution of the von Mises criterion signed with the sign of the trace of the Cauchy stress tensor. The maxima in the stiffener are of 44,5 MPa.

The dominant principal stress reaches 53 MPa which is above the class of the detail of only 36 MPa for such fillet welds.

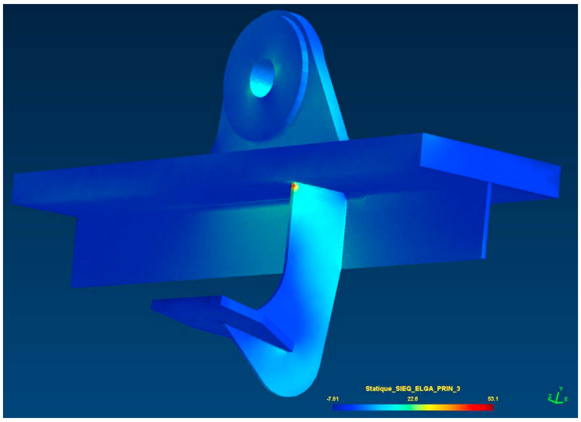


Fig. 23 : Principal stress n°3

5. Case of an old motorway bridge

Under very similar conditions, the fillet welds of a motorway bridge recently presented cracks after two decades of traffic under the effect of the distortion of the box girder and of the local bending of the concrete deck under the weight of the trucks. This location B was under surveillance as it was explained in a “Fatigue design 15” article **[12]** and the first crack of type B was discovered at end 2015. Spectacular crack of type A was already presented in **[12].**

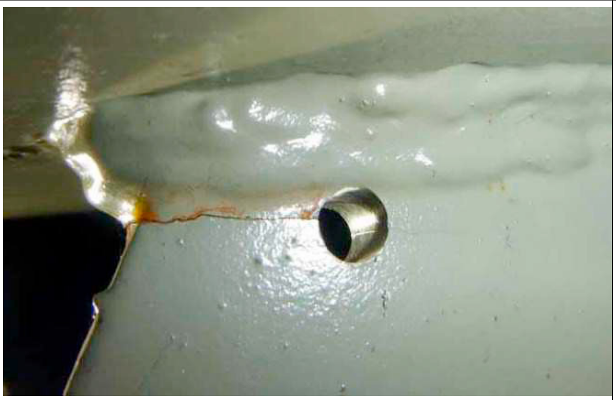


Fig. 23 : Crack at a stiffener attachment at B

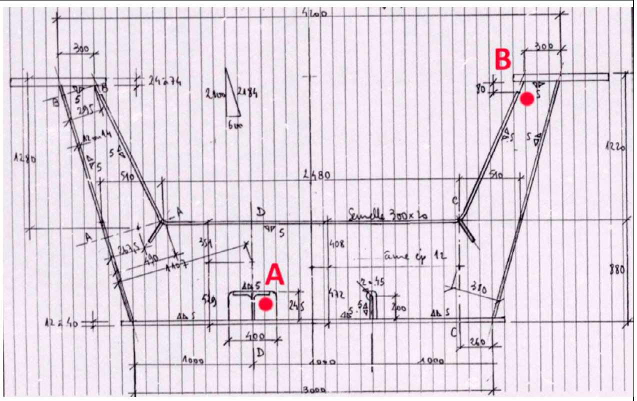


Fig. 24 : Location of point B of the crack in figure 23

6. Conclusion

Details that may appear as accessory to most of the usual bridge designers may be in fact of a crucial importance. It is in particular the case of the welded joints.

The evaluation of welded joints with fillet welds is now possible with the recent possibilities of the Finite Element Method. But such studies are very complicated and take a long time. The fillet weld joints are much more sensible to the fatigue stresses than butt welds. They should be avoided when there is a single doubt.
In addition to the fatigue problem presented in this article, the steel Z_{ed} quality of the plate which is in tension through its thickness between both welded attachments must be chosen according to the NF EN 10164 and it is recommended to carry out in addition ultrasonic tests at the location of the welds: the aim is to avoid the risk of failure due to delamination.

If the designer of a bridge uses fillet weld joints in the project or if he does not clearly specify the solution for the attachments, he has to verify the relevance of such fillet weld attachment regarding fatigue before the call for tender, because the time for such studies is generally not available during the execution studies. For all these reasons it is recommended to avoid such welds and to drive the tension attachment directly to the webs through an opening of the flange.

Acknowledgements

Acknowledgements to the owners of the concerned structures who accepted the publication of details for scientific evaluation and knowledge diffusion.

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