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# IMPROVED FATIGUE ASSESSMENT TECHNIQUES OF CONNECTING WELDS IN ORTHOTROPIC BRIDGE DECKS

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## INTRODUCTION

Although fatigue in steel structures is the most important type of fracture, because of its complexity it is less understood than other types of failure. In the past, fatigue problems were sometimes overlooked during design. With the current design codes, a fatigue problem is assessed based on SN-curves. However these curves should be updated for every project where a different design approach or installation procedure is used. This mostly not being the case, a misunderstanding of the fatigue behaviour of the detail occurs. In addition, the Palmgren-Miner method is used to calculate the lifetime of each detail. But this method is not very accurate because the load history and the load sequences do not have any effect on the fatigue resistance. These design imperfections lead to overestimating the dimensions when considering orthotropic bridge deck plates.

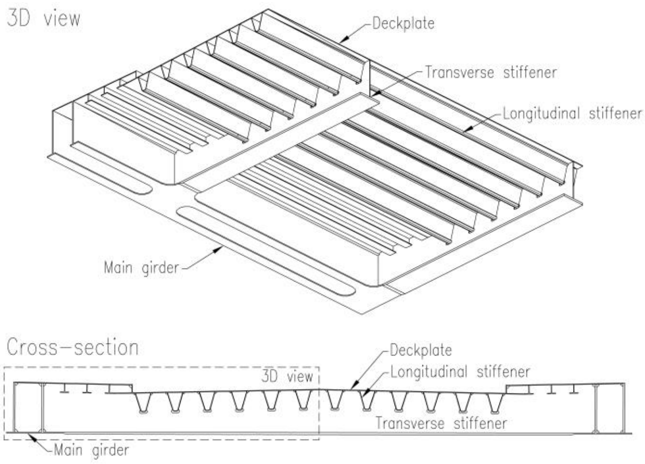


*Fig. 1.* Cross-section and 3D view of an orthotropic bridge deck

Orthotropic bridge decks are widely used in long span steel bridges since they are extremely light weighted when compared to the load carrying capacity and are therefore durable and very efficient. These types of bridge decks consist of a complex network of closed trapezoidal longitudinal stiffeners and transverse web stiffeners welded to a deck plate (*Fig. 1*). Because of the latter, various complex welding operations cause multiple fatigue problems across the bridge deck. To investigate the complex fatigue behaviour, other methods are needed besides the traditional way of fatigue calculation. Therefore, a more in-depth analysis is used: fracture mechanics. With this method a detailed crack behaviour can be evaluated. Also, a much larger dataset will be available for estimating the total fatigue lifetime or even a more realistic evaluation for the remaining lifetime of existing bridges.

## 1 DESCRIPTION OF THE PROBLEM

In the past 10 years, several fatigue problems have been observed in several orthotropic bridge decks across Europe. Especially in the Netherlands, where many of these bridge decks were constructed [1]. An example in Belgium described in this article, is the Temse Bridge across the river Scheldt (*Fig. 4*, *5*). Fatigue problems are mostly due to the lack of understanding of the fatigue behaviour. Additionally, the traffic intensity and also the traffic loads increased a lot since the construction of these bridge decks. But even recent constructed bridges could develop fatigue cracks [1]. This confirms that a more in-depth analysis is needed. In addition, fatigue cracks often initiate at the weld root (*Fig. 2*). So, in the case of a stiffener-to-deck plate connection with closed stiffeners, the crack would propagate without any possible detection through visual inspection. Therefore, the fatigue crack would only be visible when there is already considerable damage (*Fig. 3*).

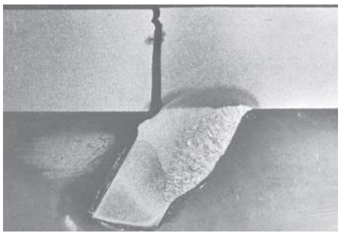

*Fig. 2.* Longitudinal crack through the deck plate at a stiffener-to-deck plate connection [2]

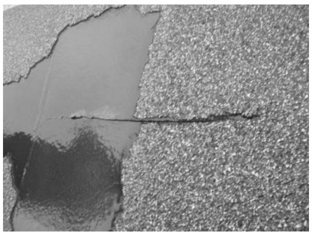

*Fig. 3.* Longitudinal crack through the deck plate at a stiffener-to-deck plate connection

## 2 FINITE ELELMENT MODELING

A FEM-model has been developed to evaluate the stress distribution and its fracture mechanics parameters in a stiffener-to-deck plate connection with closed stiffeners. Therefore, the model has been made according to the movable part of the Temse Bridge across the river Scheldt in Belgium (*Fig. 4*, *5*). After the renovation of the deck in 1994, a 600 mm long crack was detected in 2004 in the stiffener-to-deck plate detail mid-span between two crossbeams. In this particular case, the crack propagated through the deck plate starting from the weld root.

*Fig. 4.* The Temse bridge across the river Scheldt (Belgium)

*Fig. 5.* Orthotropic deck plate of the Temse bridge

The deck plate of this movable part of the bridge is 12 mm thick and the stiffeners are 8 mm thick. The overall dimensions of the traffic lane are 53,90 m by 7,00 m. The trapezoidal stiffeners are 350 mm high and 300 mm wide on top and 90 mm width at the lower soffit. The distance between the longitudinal stiffeners equals 300 mm.

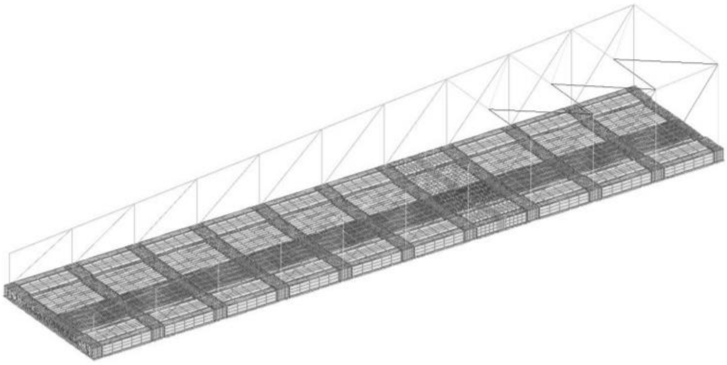

*Fig. 6.* FEM-model of the Temse Bridge

*Fig. 6* illustrates the FEM-model consists out of shell elements and beams. By doing this, realistic boundary conditions are available without seriously increasing the computation time. However for evaluating fracture mechanics parameters, a 3D model with volume elements becomes necessary. Therefore, a more detailed small-scale model is constructed (*Fig. 8*). To relate both models, the displacements are taken from the large model and introduced on the boundaries of the small-scale model. Concerning the applied stresses, load model 4 is used according to the Eurocode 1 [3]. This implies that a traffic distribution is used with five characteristic lorries. No eccentricity of the axle loads is taken into account with respect to the mid-point of the traffic lane.

### 2.1 Implementation of residual stresses

Residual stresses are present in many civil structures due to manufacturing actions causing plastic deformations. Nevertheless, these stresses are not often taken into account when considering the design of these structures. This is true when only focussing on the stress variations which eliminates the initial stress state of the structure. However the effect of residual stresses may either be beneficial or detrimental, depending on magnitude, sign and distribution of these stresses with respect to the load-induced stresses [4]. Therefore, the initial stress state due to a welding operation is introduced into the FEM-model. Basically there are two different methods to introduce an initial stress state into the model. The easiest and preferred way is to apply the residual stresses according to literature or test data. This can be done by imposing the stresses directly into the model or by imposing complementary normal forces and bending moments.

The latter is used in this paper. Results from similar fillet welds as those in the orthotropic bridge decks are used. Therefore, $N_{deck}$ is chosen in order to have tensile yield stresses into the deck plate at the weld. For the stresses into the stiffener, an additional bending moment $M_{stiffener}$ and normal force $N_{stiffener}$ are also introduced. The bending moment is necessary because the weld is welded from one side only and the filler metal and the corresponding heat zone is larger at the weld toe compared to the weld root. For the magnitude of this bending moment and normal force, an assumption is made based on the distribution of the filler metal.

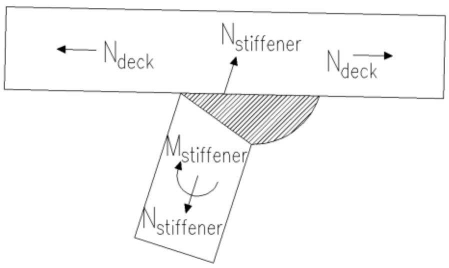


*Fig. 7.* Complementary normal forces and bending moments to simulate residual stresses

A second method to introduce residual stresses into the model is to simulate the welding operation with a finite element model [4, 5]. In this case, a thermal flux is simulated according to the real welding procedure. The result of this time dependent thermal load can be used as an initial load. This method however requires a 3D model. This is also a more advisable method because there are many uncertainties due to the lack of test data for this type of weld.

## 3 LEFM DESIGN APPROACH

Linear Elastic Fracture Mechanics (LEFM) calculations are carried out with the detailed 3D-model of a stiffener-to-deck plate connection mentioned before. The method described below refers to the automatic crack propagation method based on XFEM (eXtended Finite Element Model) techniques. With this method, it is possible to evaluate the whole crack propagation without re-meshing the model for every crack propagation step. In addition, not only the crack growth rate could be evaluated, but also the crack growth direction.

At first, an initial crack length should be chosen according to the welding detail and construction technology. Often an initial crack length is chosen between 0.1 and 1 mm [6]. If the weld is perfectly accessible to smoothen the surface afterwards, the initial crack length can be very small. However, the welds used for longitudinal stiffeners in orthotropic plated decks are welded from one side only and even the lack of penetration can be questioned. Therefore, due to the large uncertainties, an initial elliptical crack length of 1 mm in longitudinal direction is assumed and 0,5 mm in transversal direction. This was also confirmed in a microscopic study of the present weld details of a stiffener-to-deck plate connection [7]. Although the fatigue crack often propagates through the deck plate, the initial crack length is chosen parallel to the deck plate and at the weld root. After implementing an initial crack length into the model (*Fig. 8*), the XFEM calculation can be operated.

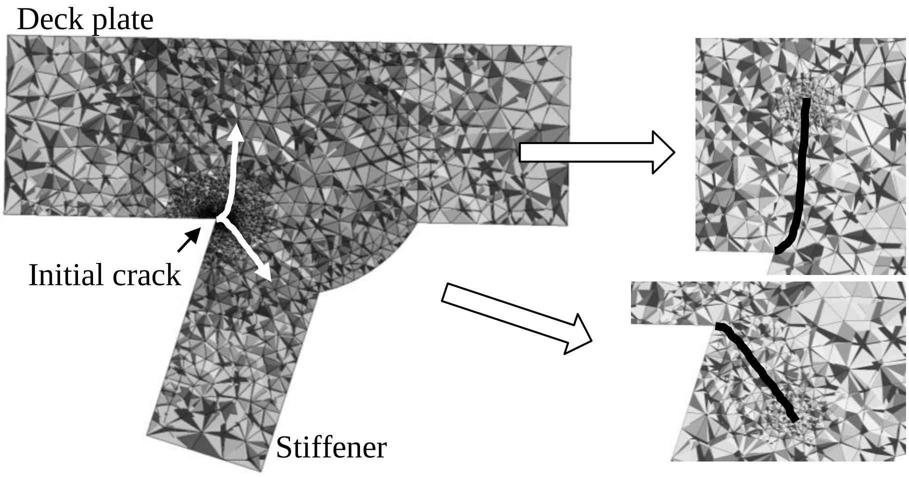


*Fig. 8*. Detailed small-scale 3D model: possible crack growth directions

The XFEM-model uses the Paris law in order to propagate the crack automatically according to the path which uses the least energy to crack:

$$\frac{da}{dN} = C \cdot \left(\Delta K_I^{eff}\right)^m \tag{1}$$

The Stress Intensity Factor (SIF) $\Delta K_I^{eff}$ in this equation is a function of $\Delta K_I$, $\Delta K_{II}$, $\Delta K_{III}$, $\theta_p$ (bifurcation angle) and the used material. The parameters C and m are material properties. For structural steel C equals $3.10^{-13}$ [N, mm] and m equals 3 [-] [1]. For the assessment of existing (old) steel bridges, C = $4.10^{-13}$ and m = 3 could be used [8]. Because the applied stress $\Delta\sigma$ is known as well as the calculated SIF-values, the geometrical dependent parameter f(a) can be evaluated from *Eq. (2)* for every crack propagation step:

$$\Delta K_I^{eff} = f(a) \cdot \Delta\sigma \cdot \sqrt{\pi \cdot a} \tag{2}$$

The geometrical parameter f(a) not only depends on the crack length a, but also on the overall dimensions of the bridge deck. Therefore, once this parameter is evaluated for a particular bridge deck geometry and weld detail, this could be used for several other bridges with comparable dimensions. *Fig. 9* illustrates the geometrical parameter f(a) for the three different wheel types used for load model 4 in Eurocode 1 [3].

According to the placement of the truck at the centre of the traffic lane, all the wheel loads are on the right hand side of the detail being considered. It can be noticed that the location of the wheel loads not really matters for the crack propagation. Only the applied stress is important. Obviously, when the wheel loads are on the left side of the detail, the stresses have an inverted sign and will create a different curve.

*Fig. 10* illustrates the geometrical parameter f(a) for the model with and without residual stresses. Only wheel type B is taken into account. It can be noticed that the curves for the model with residual stresses are shifted upwards resulting in a faster crack propagation. This is remarkable because the applied stress $\Delta\sigma$ remains identical. This inflects the importance of the initial stress state in fatigue calculations. It is also logical that a fatigue crack propagates faster if residual stresses are included. Without residual stresses, the wheel loads create a compression zone at the weld root. Therefore, the crack propagation is due to compressive forces. Including residual stresses which equal the yield tensile stress, a tensile stress variation is created.

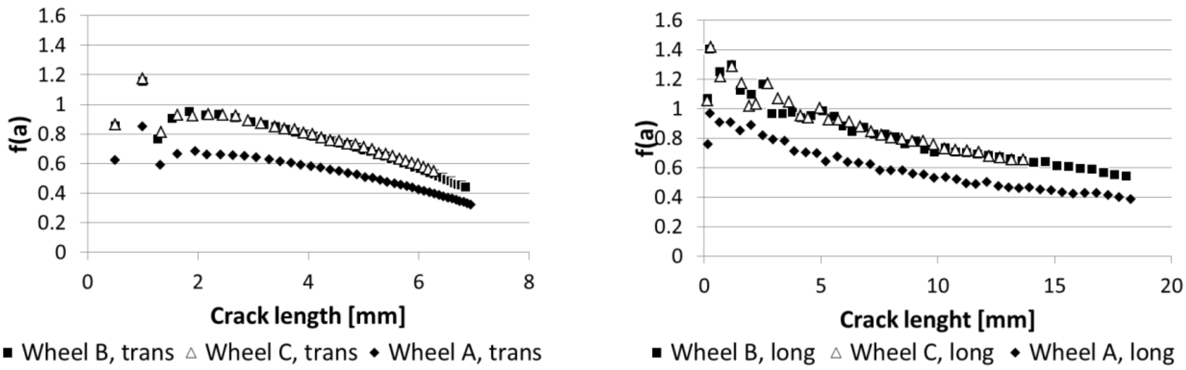


*Fig. 9.* Geometrical parameter f(a) for the three different wheel types according Eurocode. Left: transversal crack growth direction. Right: longitudinal crack growth direction

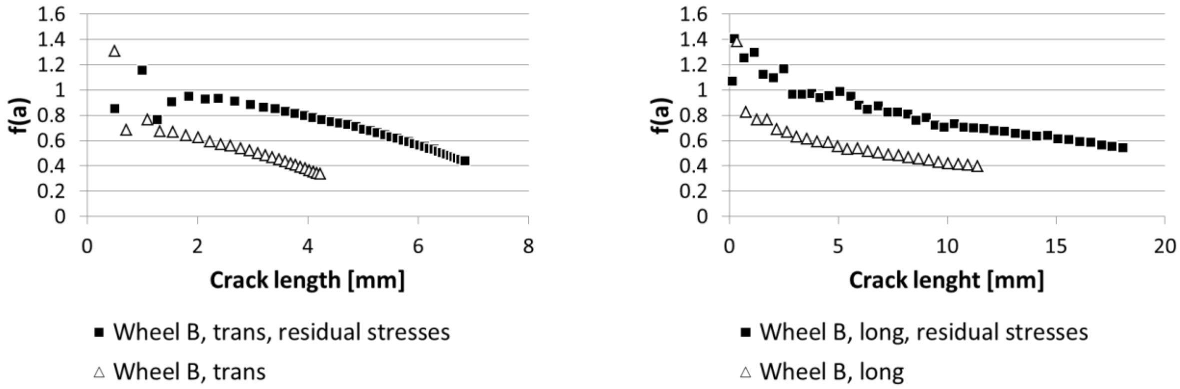


*Fig. 10.* Geometrical parameter f(a) for wheel type B: the influence of residual stresses. Left: transversal crack growth direction. Right: longitudinal crack growth direction

Another result concerns the crack growth direction. Without residual stresses the crack is growing through the stiffener web (*Fig. 8*, bottom right). If residual stresses are included, the crack grows through the deck plate (*Fig. 8*, top right). This also explains why the SIF-values are higher because the applied stresses in the deck plate differ from those in the stiffeners. It demonstrates that the simulation needs to include residual stress to comply with the real pattern of *Fig. 3*.
Finally, the fatigue life of the structure can be evaluated by integrating *Eq. (1)*:

$$N_f = \int_{a_i}^{a_f} \frac{da}{C \cdot \Delta K_I^{eff\,m}} \tag{3}$$

*Fig. 11* and *Fig. 12* visualize the evolution of the crack length as a function of the years of service life for both the transversal and longitudinal crack growth directions. At this point, the fatigue life is evaluated due to a constant stress amplitude with wheel type B and an axle load of 130 kN. A fatigue evaluation with a realist traffic load distribution will be done in future research. Without residual stresses, the crack does not develop really fast but remains faster than the crack with residual stresses until approximately 52 years. After that, the crack with residual stresses grows progressively. Because the crack is growing through the deck plate, the continuity of the stress distribution due to membrane forces are interrupted. The stresses are forced into the less rigid body of the closed stiffeners. This explains why the crack propagation through the deck plate is much faster than the crack propagation through the stiffener.
The same conclusions hold for the longitudinal crack growth direction. Although it should be noticed that the speed of the longitudinal crack growth is much faster than the one in the transversal

direction. This is identical as the fatigue problem detected in the Temse Bridge. The crack first grows longitudinally before fully penetrating the deck plate (or the stiffener). Therefore, the crack stays invisible through visible detection unless there is already sufficient damage.

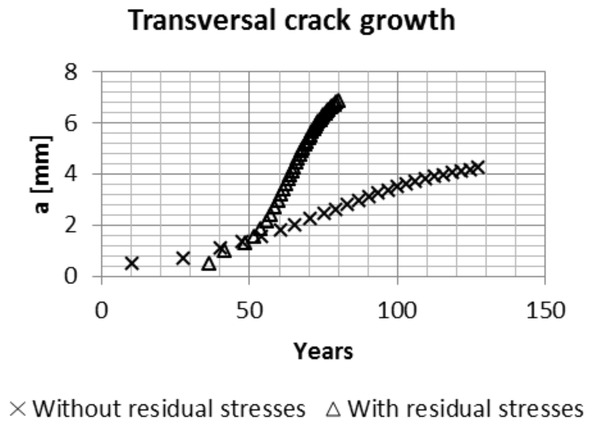


*Fig. 11*. Transversal crack growth: a comparison with residual stresses or without them

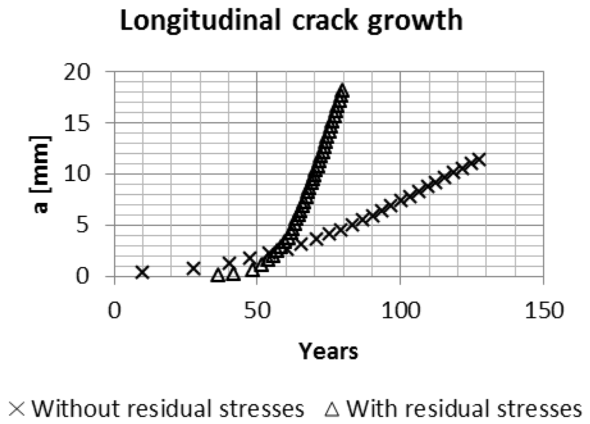


*Fig. 12*. Longitudinal crack growth: a comparison with residual stresses or without them

## 4 CONCLUSIONS

Although residual stresses are not often taken into account in the design codes, they are of capital importance for both the fatigue life of the structure and the crack growth direction. Sometimes the initial stress state can increase the crack growth speed which can never be detected with traditional fatigue calculations. It becomes clear that fracture mechanics as an improved fatigue life assessment in orthotropic bridge decks has many advantages. Every weld detail can be evaluated in depth with much more available data with respect to the traditional fatigue calculations. In addition, a geometrical parameter f(a) makes it possible to evaluate similar weld details without an intensive XFEM calculation.

Application of this method to the case of the Temse Bridge in Belgium results in a crack propagation very similar to reality. However, the stresses are still lower in the model than reality. This explains why the fatigue life in the model is still overestimated. In addition, the weld connection is not as smooth as modelled. These irregularities cause higher stress peaks. In addition, a transversal deck plate weld interferes with this connection and causes higher stress peaks. More detailed verifications are still in progress and will be published in future papers.

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