FATIGUE DAMAGE AND REPAIR OF AN ORTHOTROPIC PLATED BRIDGE DECK AFTER 23 YEARS OF SERVICE

Philippe Van Bogaert[1]

Civil Engineering Department, Ghent University (Belgium)

Abstract*: The original longest bridge across river Scheldt was designed by Gustave Eiffel. After its demolition during WW II hostilities, it was rebuild in 1955 as a truss bridge with movable part. After this, the movable part was already replaced twice, mainly due to fatigue damage. Recently severe and general damage was detected, mainly by time of flight diffraction method. This damage is initiated at the stiffener to deck plate welded joint and cracks are running rapidly through the deck plate. Subsequently extensive repair has been undertaken, extracting many parts of the upper plate and rewilding them to the stiffeners. However, in the near future further damage may be expected and plans are being made to replace the movable part of the bridge once again. Simulations show that the repeated damage may well be due to poor welding quality and the new bridge span should be have higher deck plate thickness, and may be a similar structure to the existing one. By doing this, the historic structure, inspired by its predecessor from Eiffel's time, may be preserved for the future.*

Key words*: Fatigue damage, orthotropic plated deck, crack repair, movable bridge, replacement bridge deck*

1. Introduction – River Scheldt bridge history

The last bridge crossing before the sea of river Scheldt is located at the town of Temse (Belgium). Other crossings downstream are tunnels. The first bridge built in 1870 at this location was designed by Gustave Eiffel [1] and carried a single railway track, as a part of the connection from Mechelen (Belgium) to Terneuzen (Netherlands) (Fig. 1). The bridge also allowed pedestrians and even cattle to cross the river. During hostilities in 1914 the bridge was seriously damaged by French and Belgian troops and was swiftly repaired by occupant force. Again, in 1940 the bridge was completely demolished by retreating armed forces. After WW II, in 1955 a new bridge for single railway track and for 2 road lanes was built as truss girders (Fig. 2). The total length of the bridge reaches 365 m, including a movable part as rolling bascule bridge of 50 m span. The nearby shipyard 'Boelwerf' replaced the movable part in 1974 [2] by a 60 m long movable deck with an aluminium deck. This allowed building larger sea vessels, sent to the Antwerp port and the sea. For many years, the aluminium deck showed cracks, which were regularly repaired. In

[1] Philippe Van Bogaert, MSCE, PhD, Em Sr Full Professor, Technologiepark 904 B 9052 Gent (Belgium), Philippe.vanbogaert@ugent.be

1994, the owner had the movable part be replaced by a steel orthotropic deck (OSD) with closed stiffeners.

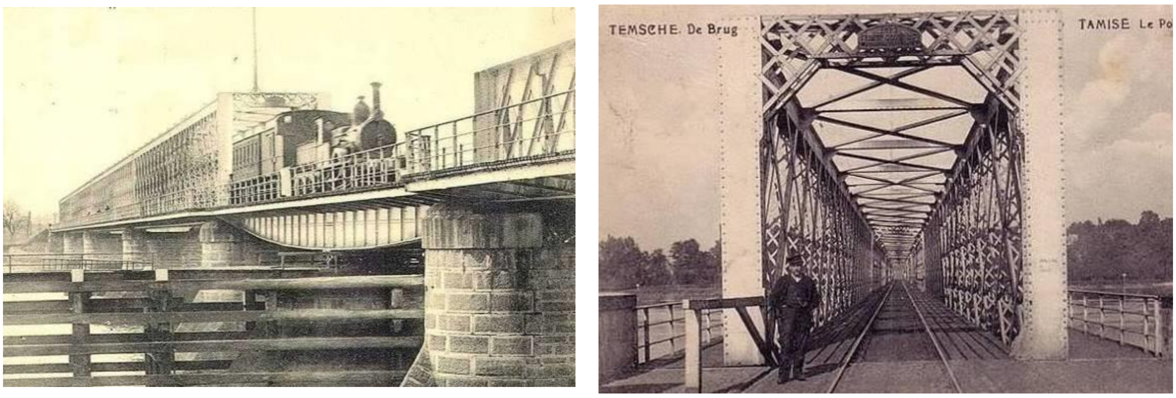

Fig. 1. River Scheldt bridge 1870 designed by G. Eiffel

Fig. 2. Overview of truss bridge built 1955

2. Fatigue damage after 10 years of service

Already in 2004, after 10 years of service, the OSD showed rapidly increasing cracks [3] at the connection of closed-section stiffeners and the deck plate. A rapid repair method was implemented. Fig. 3 shows the crack through the surfacing. Obviously the crack grew from the right side of the picture to the left. At the time, the writer was contacted, the crack through the 12 mm thick steel deck plate had grown already to a length of 600 mm. Inspection of the crack surface was impossible, although one could suspect that fatigue was the cause of such a rapidly growing crack.

As the lower side of the bridge deck was inspected, the initiation of the cracks appeared to correspond to the location of the connection of the closed stiffener to the deck plate, more precisely to the intersection of this weld with the butt weld in the stiffener as shown in Fig. 3. The stiffener to deck plate connection is known to be a detail, particularly prone to fatigue. If this fatigue detail is combined with the transverse butt weld a particularly heavy stress concentration is introduced. As can be noticed from Fig. 3 none of the butt welds received post-treatment such as grinding, as should certainly be applied in orthotropic plated bridge decks. It was immediately suspected that this particular situation was the cause of the rapidly growing crack.

Fig. 3. Crack in deck plate (left) interference with stiffener butt weld (right)

Simultaneously, numerical simulations proved that cracking was due to fatigue of the stiffener to deck plate welding and that the 10 year life time could easily be explained. Due to the intersection of welds, the stress at the connection itself is increased by the butt weld by a concentration factor of 1.19. Hence, the problem was local only and further general cracking was not expected immediately. However, the general condition of the OSD appeared rather poor and the prediction was that fatigue cracks of the type of Fig. 3 would become general within 6 to 10 years. The present paper reports on the fatigue damage that occurred, 14 years later and recent measures that were taken.

3. Fatigue resistance of stiffener to deck plate joint

Before reporting on further damage to the River Scheldt bridge, in this section, some considerations are made concerning the type of fatigue fracture found in this bridge. These include strategies to determine the fatigue life.

Apart from the connection of closed section stiffeners to the webs of crossbeams, the welding of the stiffeners to the deck plate is highly prone to fatigue cracking. Extensive research, including more recently using fracture mechanics and residual stresses [4], has been dedicated to this weakness of OSD. Eurocode EN 1993-1-9 [5] is rather strict on this, imposing category 71 MPa. More extensive research [10] has demonstrated that the category may well be 100 MPa or higher. For this connection, it is rather difficult to derive nominal stress as considered by Eurocode, especially if a FE-model is used.

In these cases, the hot spot method may be used according to [5] and further commented in [6]. Hot spot stress equals the nominal stress times the stress concentration factor, due to geometrical discontinuity or minor misalignments. This means the effect of material induced fatigue cracking is separated from the geometrical parameter. As a result, the number of fatigue design classes is reduced considerably, since only the material initial cracking effect, mainly due to welding, is considered. For most welds, the hot spot fatigue class 100 MPa applies.

However, in view of the effective crack pattern (discussed hereafter), and the fact that the hot spot stress method applies to but welds and weld-toe initiated cracks, the application to the stiffener to deck plate joint is uncertain. In addition, numerical simulation of the hot spot stress strongly depends on the FE-mesh refinement.

In [6] linear extrapolation to a weld toe from values obtained at 0.4 t and 1 t distance from the weld toe is allowed. This is sometimes refined for calculated values as a function of the dimensions of elements.

As the most critical section of the stiffener to deck plate connection is located at the root of the welding, shown in Fig. 4. This is why there is no direct evidence that the extrapolation method for hot spot stress may be copied for this particular welding detail. However, the latter is frequently assumed, also in the former paper on fatigue damage of the River Scheldt bridge [3]. Fig. 4 left shows a macrographic photo of the typical one-sided welding connection of the stiffener to the deck plate, whereas the right hand side displays the orientation of the fatigue crack, originated at the weld root and progressing to the top of the deck plate. The shaded area in the figure corresponds to the heat affected zone.

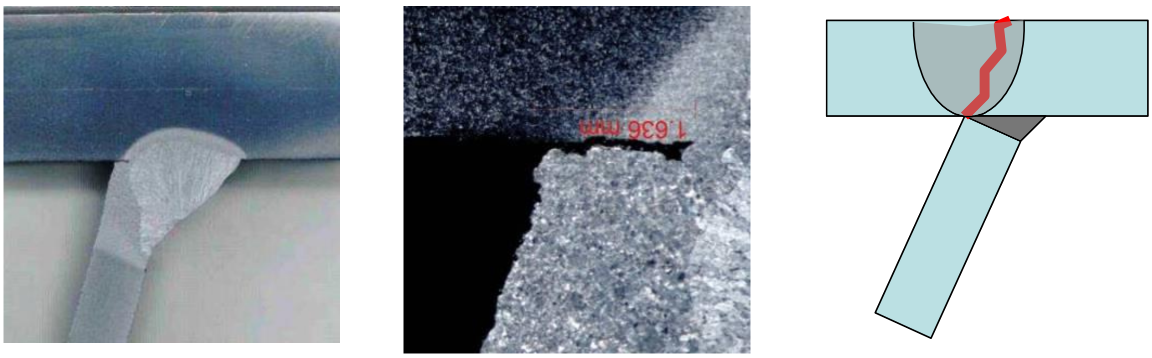

Fig. 4. (left) single side welding (centre) detail of stiffener (right) fatigue crack path

Obviously, the numerical modelling of this connection must include the small gap or lack of welding penetration. Most simulations introduce such a gap, including a round tip at the end. Microscopic research has proven that such a fillet rounding is close to reality. However, there is little difference in fatigue strength between a connection of 20% lack of penetration and a full penetration welding. This is due to the high influence of residual stress at this location. The crack path of Fig. 4 right can only be found numerically if the effect of residual stress in included.

4. Recent fatigue damage after 22 years

4.1. Non-destructive testing

It is well-known that classical US-testing does not apply to fillet or partial penetration welds, since these show gaps, which are detected as errors of quality defects. US-testing of the weld of Fig. 4 would automatically result in its rejection, as the penetration is incomplete. However, the knowledge of the lack of penetration is imperative.

Apart from this, as the fatigue-induced cracks grow from the weld root towards the top side of the deck plate, nor the initiation and neither the growing of the crack can be measured or seen, until it appears through the bridge pavement, as found back in 2004 [3]. However, today developments in ND-testing allow assessment of the magnitude of gaps or defects. The time of flight diffraction method [7] TOFD does not use the reflection of US-waves to detect defects, but it uses deviations of the waves from two opposite sides of the weld. When a crack is present, there is a diffraction of the ultrasonic wave from the tip(s) of the crack. Using the measured time of flight of the pulse, the depth of a crack tips can be calculated automatically by simple trigonometry. The principle of TOFD-testing is shown in Fig. 5.

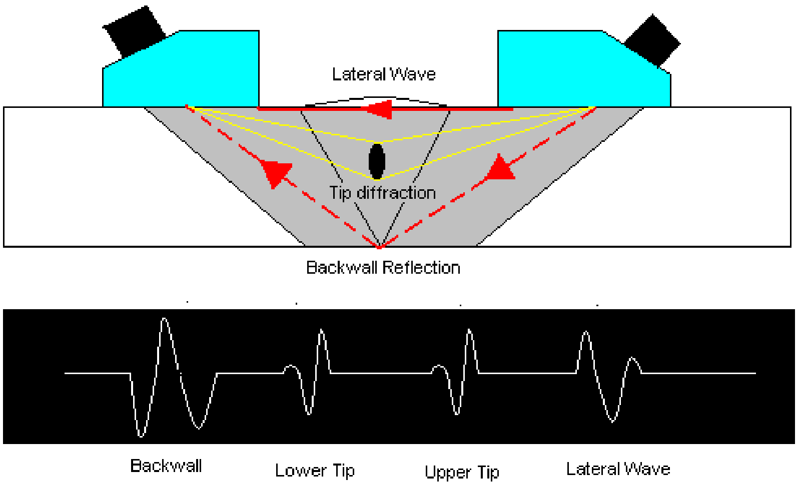

Fig. 5. Principle of TOFD testing and output signal

In 2017 TOFD non-destructive testing was carried out on the deck plate of the movable part of the River Scheldt bridge. It demonstrated that the plate was cracking rapidly, since 14 m length was seriously affected. Some cracks already were visible through the 7 mm pavement of the bridge deck, and there was a high probability that more in depth cracking was initiated or already. As these cracks are invisible, extensive testing became necessary. The owner decided to have these parts of the deck plate repaired, according to a procedure involving the replacement of the plate from above and rewelding of the connection of Fig. 4 [8] from below the bridge.

4.2. Extensive repair

The repair method may best be illustrated through the pictures of Fig. 6. Vertical plates, are bridging the cracked part of the deck plate. This allows to burn the circumference of the plate area and to cut the welds of the connection of Fig. 6. This condition is shown in Fig. 6 left. Fig. 6 right shows the precise cut in the deck plate.

Fig. 6. Removal of cracked parts of deck plate

Subsequently, a new part of deck plate is being placed and welded to the free edges. As for the connection to the closed stiffeners, this is welded from below, using a suspended rack. The replacing deck plate is increased from 12 to 16 mm. Hence, a higher resistance for bending is obtained and cracking will become less critical.

Fig. 7. Repaired deck plate areas

During repair works, the ND-inspections were continued and the plate length to be substituted increased from a presumable value of 14 m to 88 m. The overall and more detailed view of Fig. 7 shows the numerous areas of the deck plate that were replaced and strengthened, as well as the finishing to avoid new crack initiations.

As the number of closed-section stiffeners equals 12, the total length of the stiffener to deck plate welds equals 2*720 m = 1440 m. However only 1/3rd of these welds is close to the circulation line of wheel loads. Hence 480 m of this type of welding is influenced, or 240 m of deck plate sections. The recent repair thus included 37% of the locations prone to fatigue damage. Consequently, the fatigue damage was general and the bridge deck was at failure condition.

As the counterweight of the movable bridge part is modified by the larger weight of the deck plate, the former has to be adapted. The original counterweight consists of a steel plated box, filled with lead blocks, both for the road and railway movable part. In 1984 the counterweight of the railway part had to be adapted because of the installation of overhead power line. During this, parts of the lead melted by the heath of cutting the plates. Hence, opening and rearranging the ballast is not advisable. Adding steel plates to the ballast box is considered as the best alternative, provided the latter do not touch the road in opened position of the bridge.

5. Future replacement of the movable part

The previous comments show that fatigue damage has become overwhelming. As a result, the owner is now considering to replace the movable part of the bridge. It is interesting to know whether an identical concept of the OSD would resist the fatigue inducing load. For this the table from [8] may be used to derive the actual fatigue resistance of the connection.

Table 1. Fatigue life time according to [8]

Category	71	90	105
Life (years) for $1.4*10^6$ cls/y	4.9	17.8	158.2

The real fatigue class certainly approaches 90 MPa. To increase the fatigue life of the bridge, the deck plate thickness should be increased to 15 mm, thus obtaining 165 years. The problem with the counterweight would be similar to the necessary adjustments mentioned in 4.

An alternative for fatigue life prediction is linear elastic fracture mechanics. This method is reputed to deliver more reliable and less conservative results. It requires the knowledge of an initial crack depth and subsequently allows calculating the progression of this crack depth, based on Paris' crack propagation law. In the expression (5.1) a is the

$$\frac{da}{dN} = C(\Delta K)^m \tag{5.1}$$

crack depth, N the number of cycles C a constant and ΔK an effective stress concentration factor. The latter may be replaced by a function of the moving value of the already reached crack depth f (a). This function must be determined experimentally. Expression (5.1) may then be replaced by the sum

$$a = \sum_i C\,(f(a_i)\;\Delta\sigma_i\,\sqrt{\pi a_i}\,)^m\,dN \tag{5.2}$$

Using the value of f(a) derived in [8], which include the effect of residual stress, the crack depth can be calculated. Obviously, the value of the initial crack depth is a dominant factor. The graphs of Fig. 8 show the result, to the left for a 12 mm thick deck plate and to the right this plate is increased to 15 mm.

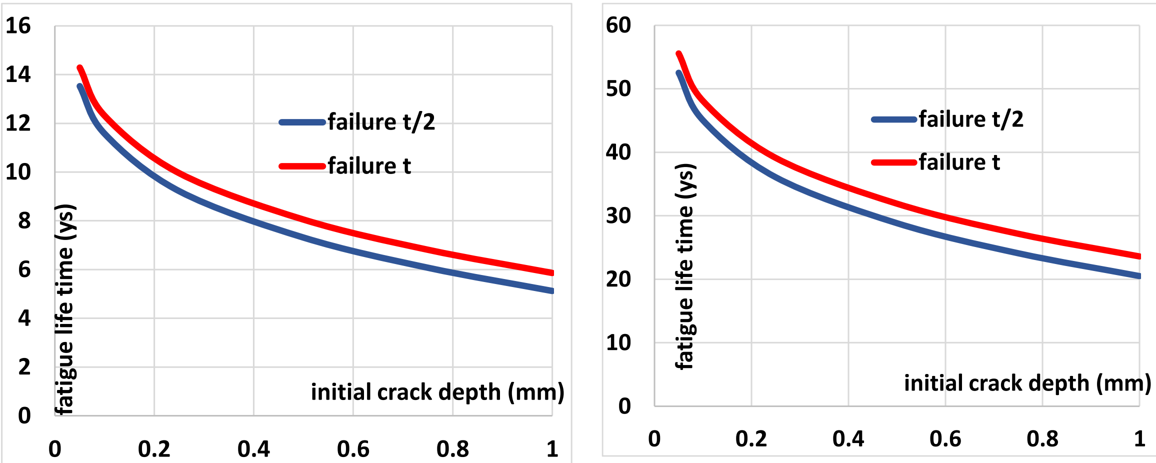

Fig. 8. Fatigue life prediction by LEFM

The graphs of Fig. 8 each show 2 curves, the failure point being an uncertain condition. Generally, failure is considered to occur when the crack depth reaches half the plate thickness and the curve t/2 is relevant. From the tests reported in [8] the initial crack depth is close to 0.1 mm. In that case, the result of LEFM is that for the initial structure fatigue life would have been 13.53 years, whereas if the deck plate thickness is increased to 15 mm it would become 52.53 years. The latter does not comply to the requirement of

100 years. It should be noted that Paris law does not recognize a modification of m with increasing number of cycles. It is possible to include this effect, albeit the coefficients for the function $f(a_i)$ as derived, probably apply to short term fatigue only. In view of this, further extension of the research in [8] should aim at tests of longer duration.

During the aforementioned repair works, welders found that the actual joint of the stiffener to the deck plate showed less penetration than Fig. 4 left. This is based on testimony from persons and could not be confirmed directly by material evidence. However, the cut edge of the stiffener could be examined closely and the flat part of the web plate top has been measured. Details of this are shown in Fig. 9.

Fig. 9. Closer image of stiffener upper edge

The value of the horizontal flat part of the web edge varies from 4.7 to 6.3 mm, the web thickness being 8 mm. This may confirm the claim of the welders. Obviously, the flat part of the web thickness is irregular and may be influenced by the cutting itself. Nevertheless, a rather evenly distributed area was found as the pictures of Fig. 9 may display.

In an attempt to derive a fatigue class for the existing bridge, and taking into account past and present situations, the Palmgren-Miner formula seems the most accurate. The average actual number of cycles per year should be $1.05\ 10^6$, and the number of years for general fatigue damage 22.5. From these data, the actual category is derived as 89.8 MPa. This certainly is lower than the relevant values of table 1.

Fatigue rainflow count, according to EN 1991-2 [9] may further be useful in trying to evaluate the failure condition as well as the future replacement alternatives. Using FLM 4, consisting of 5 lorries and adopting the case of long distance traffic, the failure condition at 22 years corresponds to a failure value of 91.6 MPa. In this it was assumed that traffic intensity equals $1.4\ 10^6$ per year, corresponding to 2800 vehicles per day. Fig. 10 shows the nominal stress variations due to lorry 3 of FLM 4.

The failure value of 91.6 MPa does not correspond to a fatigue class as in the code, since it does not include any statistical parameter. In addition, no factor γ_{Mf} is included. Thus it cannot be used as a design parameter. It does however correspond closely to the value 87 MPa as mentioned in [10]. According to this reference, the design stress range would be 70 MPa, whereas the code classifies this as 50 MPa. Clearly, this welding quality is not suited for a bridge deck with heavy road traffic. A similar rainflow fatigue count shows that the design stress range should equal 111.3 MPa, in order to obtain a fatigue life of 100 years.

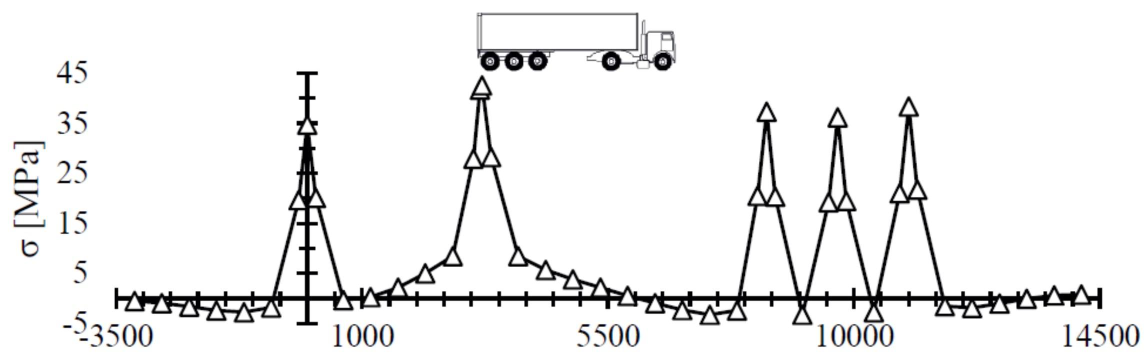

Fig. 10. Stress variation caused by lorry from EN 1991-2

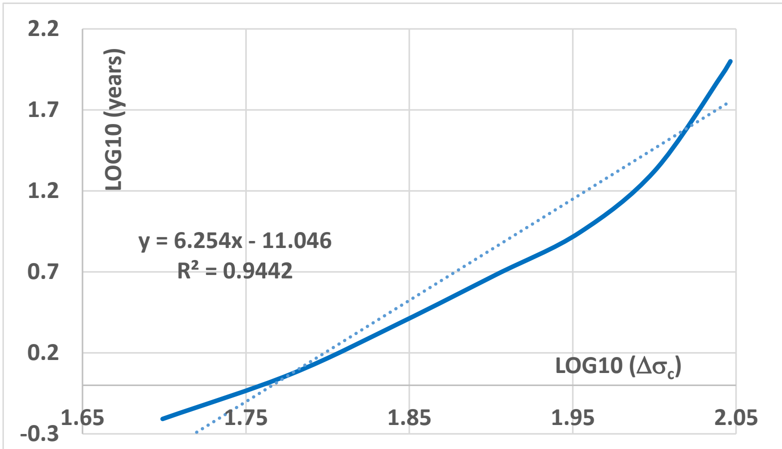

Fig. 11. Relation of design fatigue category and fatigue life

The process has been repeated for various values of the design stress variation, resulting in the bi-logarithmic graph of Fig. 11. A linear relation may be expected, although the ascending curve may be explained by the exceeding of the cut-off limit. Since the required design category needed to obtain 100 year fatigue life equals 111.3, no practical solution exists to replace the movable part of the bridge. Should the deck plate thickness be increased to 15 mm, the design stress category should equal to 71.3 MPa, which is consistent with the code.

Hence, the fatigue damage and subsequent general failure of the River Scheldt bridge may well be due to insufficient welding quality and the movable part may be substituted by an identical structure, provided the deck plate thickness is increased to 15 mm and partial penetration welding is used. In this manner, this historic structure, inspired by its predecessor from Eiffel's time, may be preserved for the future and standing as a landmark.

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