Fatigue assessment of an existing orthotropic steel deck to comply with future traffic intensity

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1 Abstract

Orthotropic Steel Decks (OSDs) are widely used in long span bridges because of their extremely light weight when compared to their load carrying capacity. These deck types typically consist of a grillage of closed trapezoidal longitudinal stiffeners and transverse web stiffeners, welded to a deck plate. As a result, fatigue problems occur due to the extensive use of complex welded connections. Unfortunately, fatigue effects have often been overlooked during design, which was also the case for an important multiple span box girder viaduct in Belgium (1978).

Due to a renovation program of the ring road around Brussels, the number of traffic lanes on the viaduct should be extended from three to four. As a result, questions have been raised about the current structural health of the OSD due to fatigue and future fatigue damage accumulation. Therefore, extensive FEM analyses have been performed, taking into account various parameters such as increased traffic volume and accompanying axle loads, historical positions of the heavy lanes, historical road pavements and their temperature-dependent load spreading effects. In conclusion, accurate fatigue damages have been determined for all fatigue details. Therefore, focused inspections and design solutions can be provided, resulting in a durable bridge management for the next 60 years.

Keywords: orthotropic steel deck bridge; renovation; fatigue damage; asphalt surfacing.

2 Introduction

During the 70's and 80's, governments across Western Europe heavily invested in infrastructure to accommodate the sudden rise in traffic. Hundreds of bridges, tunnels and roads were constructed in a short period of time. The Orthotropic Steel Deck (OSD) appeared to be an economic solution for highway bridges. However, fatigue damage has become a wide-spread issue for this type of bridge due to the heavily increased traffic volume, the increased axle loads and especially because the assessment of fatigue was never fully considered, resulting in poor fatigue categories [1-3].

In 2018, the Flemish Government decided to strongly invest in the infrastructure of the ringway of Brussels, among which the viaduct of Vilvoorde, built in 1978 (Figure 1 (a)). This viaduct consists of a 888 m twin multi-span steel closed box girders, using an OSD. The longest span amounts 163 m.

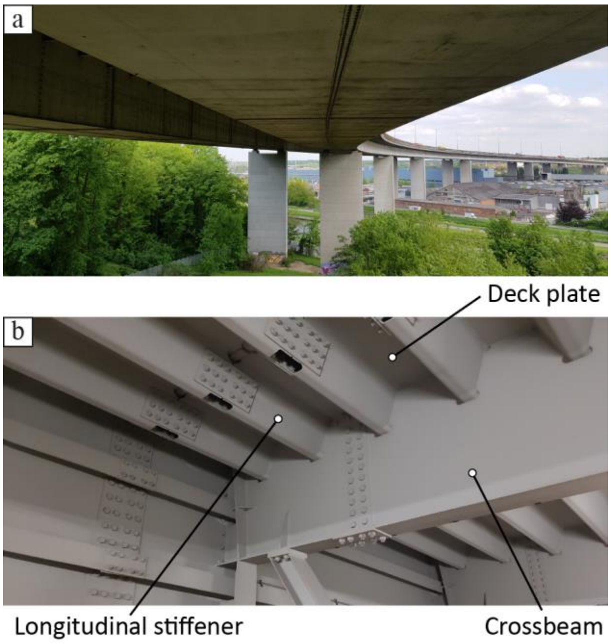

Figure 1. Viaduct of Vilvoorde. (a) Closed box girder seen from below; (b) OSD details inside the box girder

An OSD typically consists of a grillage of closed trapezoidal longitudinal stiffeners and crossbeams, welded to a deck plate (Figure 1 (b)). The viaduct of Vilvoorde uses a deck plate thickness of 12 mm and the stiffener's thickness varies from 6 to 8 mm. The crossbeams are 10 mm thick.

Due to the renovation program of the ringway, the number of traffic lanes on the bridge should increase from three to four. Questions have been raised about the current fatigue damage stage of the OSD and the impact of the renovation on the remaining fatigue life. Therefore, an intensive FE analysis was requested.

3 Common fatigue problems in OSDs

Due to the complexity and the numerous welding operations in OSDs, various fatigue problems have been reported, illustrating the necessity to assess and to control the critical fatigue areas.

The focus of this paper will be on cracks developing in the deck plate and in the longitudinal stiffeners. Especially the details at intersections and close to discontinuities are of interest, as those are more prone to fatigue [1,2]. In addition, the connection of the cantilever crossbeam is assessed, as this part will be heavily loaded after renovation. In Figure 2 and Table 1, an overview is given of the details discussed in this paper. Some detail categories are based on standards, others are based on the (re-) evaluation of fatigue test data. Fatigue details 1 through 8 are evaluated for stiffeners number one and two (see Figure 2).

4 Historical and future boundary conditions

One of the main challenges when assessing the remaining fatigue life of a structure is to determine the fatigue damage due to all historical conditions, such as the evolution of the trucks categories and their corresponding axle loads, the influence of the temperature, the use of different asphalt layers, etc. Some of these conditions are well documented, while others are very difficult to determine. Therefore, thorough assumptions have to be made, which was also the case for the viaduct of Vilvoorde.

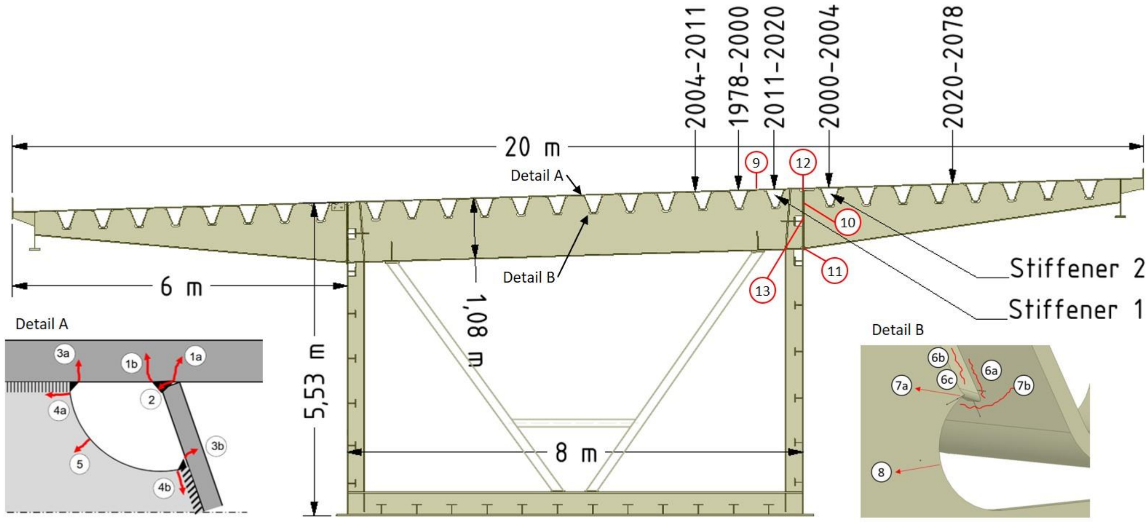

Figure 2. Global dimensions of the closed box girder including the fatigue details and historical truck positions (arrow represents the center of the right wheel of the trucks)

Table 1. List of fatigue details with corresponding fatigue detail categories. (*) These details are evaluated both at the crossbeam and in between two crossbeams.

Fatigue detail	Description	$\Delta\sigma_{c,95\%}$ [MPa]
1a(*)	Cracks in the deck plate at the weld root of the stiffener-to-deck plate joint	125
1b(*)	Cracks in the deck plate at the weld toe of the stiffener-to-deck plate joint	125
2(*)	Cracks in the weld bead of the stiffener-to-deck plate joint	90
3a/3b	Cracks in the deck plate or the stiffener web at the weld ends of the weld access hole	90
4a/4b	Weld toe crack from the edge of weld access hole into in the crossbeam	80
5	Cracks in the free edge of the weld access hole	140
6a	Weld toe crack into the stiffeners web at the crossbeam-to-stiffener connection	80
6b	Weld root crack of the crossbeam-to-stiffener connection	40
7a	Weld toe crack from the edge of cope hole into in the crossbeam	71
7b	Crack in the stiffeners web at the weld ends of the cope hole	90
8	Cracks in the free edge of the cope hole	140
9	Transverse butt weld with backing strip in the deck plate	71
10	Weld root crack in the connection of the crossbeams web to its end plate	40
11	Weld root crack in the connection of the crossbeams flange to its end plate	40
12a	Weld toe crack into the deck plate at the connection of box girders web to deck plate	80
12b	Weld root crack in the connection of the box girders web to the deck plate	40
13	Cracks in the one sided connection with preloaded high strength bolts	90

4.1 Position of the heaviest traffic lane

Since the opening of the viaduct in 1978, mainly four different traffic lane layouts have been applied, as illustrated in Figure 2. After the renovation in 2020, the trucks will shift to the right cantilever part. In addition to the position of the trucks, the transverse distribution of the trucks within its lane according to Eurocode 1991-2 §4.6.1 (5) is applied [4]. Temporary layouts for short term road works were not considered as they only have a small influence on the calculated fatigue damages.

4.2 Asphalt layers

During the fatigue design of a new bridge, the composite effect of the asphalt layers is not considered except for its load spreading effect. However, the asphalt layers can have a significant influence on the fatigue design as its load spreading effects are affected by temperature. At low temperatures, the asphalt becomes very stiff and will spread its wheel load over a large area. At higher temperatures, the asphalt becomes so weak that the wheel load is directly transferred to steel details underneath. This effect is even non-linear for the details near the deck plate surface and it is therefore important to consider multiple stress calculations for sufficient temperature conditions. In between the calculated conditions, linear interpolation is then allowed.

As for the composite effect, several studies already concluded that the membrane between the steel deck plate and the above asphalt pavement has a very low stiffness, resulting in a near shear free connection [1,5]. Therefore, the asphalt layer does not really contribute in the composite action, but only in the load spreading effects.

For the viaduct of Vilvoorde, two different asphalt layers have been used with a total thickness of 125 mm. The original one uses an open asphalt top layer, while the current top layer is based on a stone mastic asphalt.

4.3 Load model

Instead of the fatigue load models provided by Eurocode 1991-2 [4], the more realistic fatigue load model of annex A of NEN 8701:2015 is used [7].

The load model consists of a selection of truck types, similar to FLM4 in EN 1991-2, but each truck type has three different weight categories with associated fractions of the number of lorries.

To improve the accuracy of the load model for evaluating the viaduct of Vilvoorde, it has been decided to update the given distributions of the different trucks types and the corresponding truck loadings by using traffic measurements close to the viaduct. The traffic counters just in front of the viaduct and WIM measurements on representative highways such as the one on which the viaduct is located have been used to recalibrate the distribution factors of the load model of annex A of NEN 8701:2015, resulting in a distribution of the low, mean and high traffic loads for the reference year 2018 equaling 40%, 39% and 21% respectively. The distribution of the truck types V11, V12 T11O2 and T11O3 corresponds with 22%, 11%, 12% and 55%. For the reference year, the number of trucks passing the viaduct each year equals 3 000 000.

5 FE model

5.1 Boundary conditions

Given the complexity of the discussed historical and future boundary conditions, an accurate FE model had to be provided, mainly because of the interaction of the temperature dependent asphalt layers with the steel deck plate. As a result, the whole viaduct is modelled using beam elements considering the multi-span behavior of the viaduct. Only the central part of the largest span is built up using shell elements. In addition, five crossbeams or four spans between the crossbeams are modelled using shell elements. Finally, the asphalt layers on top of the steel deck plate are modelled using quadrilateral volume elements. The asphalt layer is connected to the deck plate using a frictionless contact definition.

The asphalt layers are meshed using an average size of 30 mm, which result from sensitivity analyses of the load spreading effect on the stress results. The mesh of the shell elements near the fatigue details is set to at least an average size equaling the thickness of the shell element.

Finally, only one axle load is modelled using moving wheel prints. Therefore, influence surfaces could be created of all wheel types and for all reference temperatures being considered. Later, during the post-processing, all different trucks categories can be composed using the influence surfaces of the individual axle loads. In summary, 10800 intensive calculations have been performed: 600 axle positions for three different axle types have been analyzed, combined with 2 different asphalt pavements at 3 different temperatures.

5.2 Submodelling

The large model of the viaduct is mainly modelled for accurately describing the global and local effects. Fatigue details which are not affected by local stress concentrations, weld geometries, contact definitions, etc. can usually be evaluated by the nominal stress method of Eurocode 1991-1-9 [6]. However, many critical details are close to the deck plate and are therefore affected by local effects of the weld shape and the contact stresses between the asphalt layer and the deck plate. Other details are located for example near stress concentrations at the cope hole or the weld access holes of the longitudinal stiffeners.

As a result, submodelling of the longitudinal stiffeners or the cantilever crossbeam is applied as illustrated in Figure 3. Submodelling allows for the deformations and rotations of the large model to be transferred as a boundary condition to the edges of the 3D volume model. In all submodels, the fillet welds are modelled with their weld root gap. In addition, no weld root penetration is considered. In the submodel of the cantilever part, also the effect of the high strength bolts is defined.

Finally, hot-spot stresses at the weld toes are derived from the submodels to account for the actual stress concentration effects according the guidelines of IIW [8]. This is for example illustrated in Figure 3 (a) on which the hot-spot stress nodes are highlighted for the calculation of the extrapolated hot-spot stresses at the weld ends in the weld access holes.

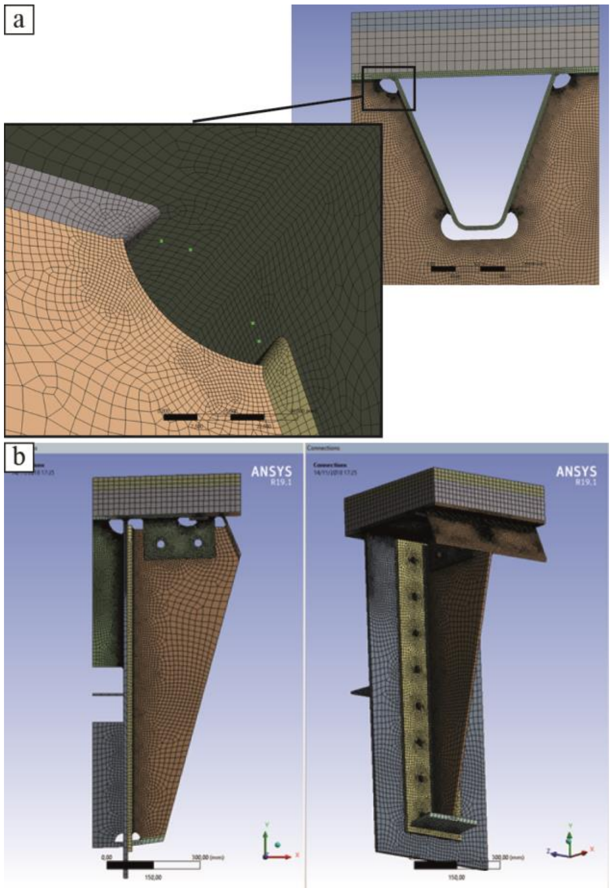

Figure 3. Submodelling. (a) Longitudinal stiffener; (b) Cantilever crossbeam.

6 Post-processing

The output of the FE model resulted in influence surfaces for each detail, such as illustrated in Figure 4 (a). In a next step, a self-developed post-processing script stochastically simulates the historical stress history, taking into account all defined variables (temperature, transverse positions, trend factors, distribution of the axle loads and truck types, etc.). As a result, stress variations are simulated (Figure 4 (b)) on which the rainflow counting is applied. Subsequently, a stress histogram is made from the rainflow counting considering the total considered lifetime of the viaduct, which is plotted against the used S-N curves in Figure 4 (c). Finally, the damage factor in function of time could be derived (Figure 4 (d)). From these graphs, it is possible to accurately define the lifetime or the time period in which the fatigue detail would possibly fail.

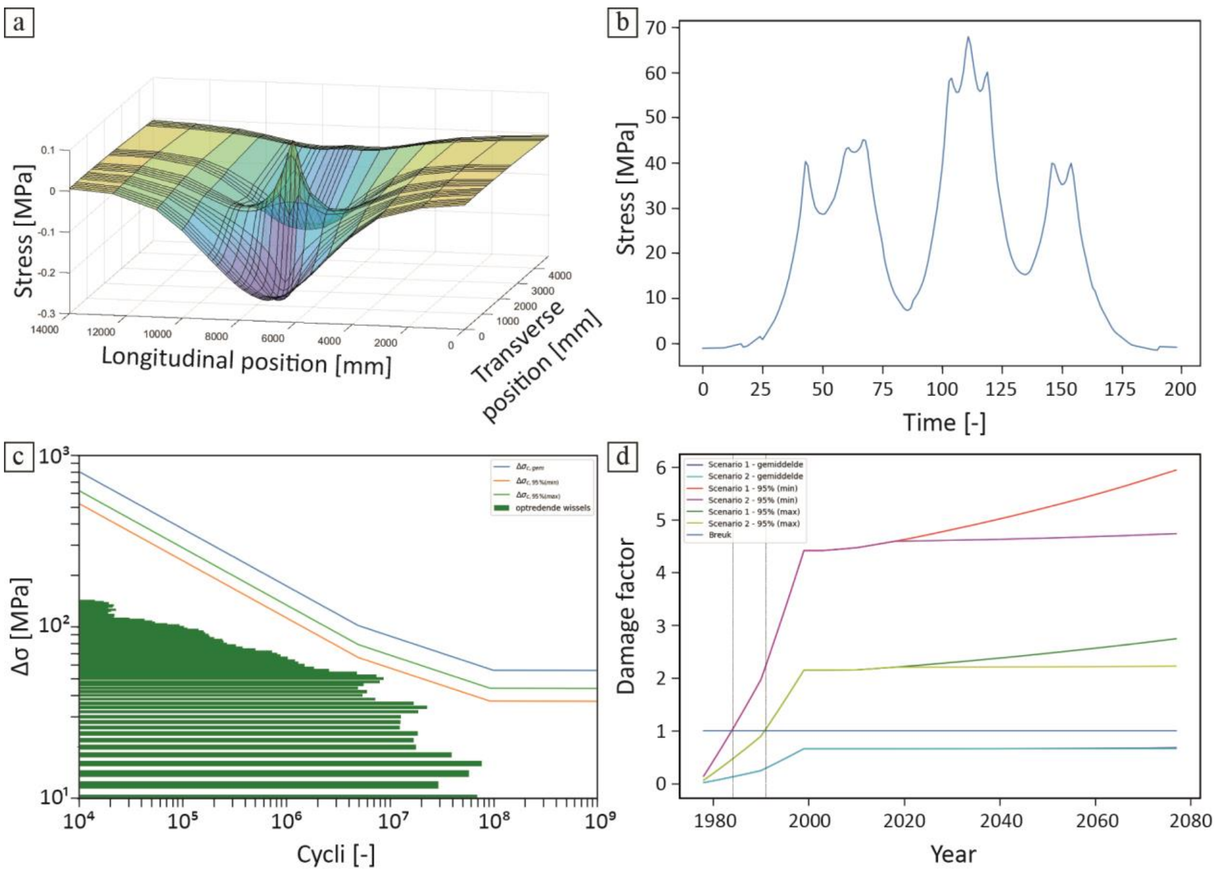

Figure 4. Post-processing flow. (a) Influence surface; (b) Stress variation due to a passing truck; (c) Total histogram of the stress cycle counting; (d) Fatigue damage curves in function of time.

7 Results

The damage factors D of all fatigue details are summarized in Table 2. In addition to the damage factor at the final lifetime of the structure ($D_{year,2068}$), also the damage factor for the current state is given ($D_{year,2018}$). From these results, it becomes clear that already some damage (D > 1) is expected in the stiffener-to-deck plate joint, details around the weld access hole and the details at the cope hole. Due to the additional traffic lane in 2020, the traffic shifts toward the second evaluated longitudinal stiffener. As a result, the damage in the second stiffener is smaller than that of the first stiffener in 2018 but larger in 2068, especially because it has to carry increasing axle loads and increasing traffic flows for a much longer period then the first stiffener. Further on, the fillet weld at the end plate will probably fail as well in the future. This could result in severe damage if not treated.

Remarkably, detail numbers 1a and 1b are not affected by fatigue, while these details often result in deck plate cracks [1-3]. The reason for the atypical low fatigue damages in these details are explained by the usage of a relative thick asphalt layer, which reduces the stresses in the steel deck.

8 Conclusions

As the fatigue assessment of the viaduct of Vilvoorde was not considered during design, several fatigue details do not fulfil the requirements of a design life of 100 years. Close to the surface the load spreading effect of the asphalt layers is beneficial. Nevertheless, the weld access holes and the cope holes give rise to stress concentrations resulting in a poor fatigue resistance. Even before the renovation program, fatigue cracks are expected.

Due to the extensive FE analysis, focused visual inspections and NDT can be performed to find out which details are actually cracked and which ones still have some reserve.

For the future lane layout, the cantilever crossbeam is longer and heavier loaded resulting in much higher damage factors. It becomes clear that due to these results and the current state of the viaduct, special care is needed to repair and strengthen the fatigue details. The advantage of such a complex FE model is that all solutions can be evaluated to meet the requirement of a design life of 100 years. In addition, the FE model can provide additional information for long term monitoring of the most critical details resulting in an optimized inspection program and therefore having a durable bridge management for the remaining lifetime.

Table 2. Fatigue damage factors D for all fatigue details given for the current state (2018) and the future state (2068). (*) Only the damage factor at location of the crossbeam is given as the ones in between two crossbeams remain zero.

Longitudinal stiffener 1													
Fatigue detail	$1a^{(*)}$	$1b^{(*)}$	$2^{(*)}$	3a	3b	4a	4b	5	6a	6b	7a	7b	8
$D_{year,2018}$ [-]	0.03	0.11	**4.59**	0.29	**3.56**	0.31	**1.46**	0.32	**6.65**	**1.62**	**19.49**	**67.25**	**21.69**
$D_{year,2068}$ [-]	0.03	0.49	**8.27**	0.34	**3.56**	0.35	**3.47**	0.82	**8.3**	**5.69**	**24.27**	**104.67**	**28.43**
Longitudinal stiffener 2													
Fatigue detail	$1a^{(*)}$	$1b^{(*)}$	$2^{(*)}$	3a	3b	4a	4b	5	6a	6b	7a	7b	8
$D_{year,2018}$ [-]	0.08	0	**2.58**	0.32	**1.09**	0.08	0.87	0.2	0.41	0.09	0.84	**1.52**	0.63
$D_{year,2068}$ [-]	**3.37**	0.05	**188.61**	**33.01**	**68.64**	**9.26**	**81.00**	**32.11**	**39.76**	**29.7**	**81.00**	**175.88**	**79.87**
Other details													
Fatigue detail	9	10	11	12a	12b	13							
$D_{year,2018}$ [-]	0	0.04	0.35	0.05	0	0							
$D_{year,2068}$ [-]	0	**33.84**	**78.98**	0.08	0.01	0							

9 References

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