FATIGUE REAL SCALE TEST COMPARISON WITH ACTUAL BRIDGE MONITORING

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In railway bridges, special attention needs to be paid to the interaction between the railway track and the bridge. Forces occur in the rails, as well as in the deck and the bearings of the bridge. These forces result in additional stresses, which need to be kept sufficiently low. Peak stresses due to e.g., temperature variations and bridge deck bending are especially to be feared at the bridge/abutment transition. Special rail fastening systems may be used to limit these additional stresses. Real scale tests were performed to verify the behavior of the track and the bridge equipped with such modified fasteners, which are designed to limit the transfer of longitudinal forces to the bridge. Based on the data from these tests, a detailed finite element model is constructed in this research paper. In contrast to the generally accepted modeling approach prescribed by international norms such as Eurocode NBN EN 1991-2 and UIC Code 774-3R, this research is performed, assuming the fact that the behavior due to track-bridge interaction can be simulated by a rail model supported by springs, i.e., without the need to model the actual bridge structure in detail. The results of these finite element model simulations are compared with data of a long-term monitoring program using strain gauges on all elements of the rails and support structure of an actual railway bridge in Belgium.

Keywords: Railway, Track behavior, Steel, Testing, Temperature loads, Finite element modelling, Rail fastening systems, Rail-bridge interaction.

1 INTRODUCTION

In Mechelen, Belgium, the railway bridges crossing the canal Leuven-Dijle are in a renovation phase. One of the most important aspects of this renovation project is the connection between the train rails and the bridge deck. Continuous welded rails are continued over a bridge with a span of 63.8 m, and the railway track and bridge are interlinked to each other by means of connections (fastening systems). Because of this interlinking, interaction between the bridge and the track will take place. When forces are applied to the continuously welded rail track, additional forces are induced into the bridge structure and/or into the bearings which are supporting the bridge deck. This goes along with movements of the rail track and the bridge deck. Alternatively, when the bridge deck moves, the rail track will tend to move as well inducing additional forces in the track (and indirectly in the bridge bearings) (De Backer et al. 2017).

A new type of connection has been used with the intention to allow a different longitudinal movement between the bridge and the train rail. This connection type should be able to limit the additional forces and stresses induced by bridge or track movement, especially at the location where the track crosses the gap in between the bridge and the abutment.

Before this new connection type was applied in the real situation, a real scale fatigue test on a bridge segment with equal characteristics was performed in collaboration with the University of Pardubice in the Czech Republic. By subjecting the train rail to horizontal and vertical loading for millions of cycles, the fatigue behavior of the test piece could be analyzed.
At the end of 2020, the actual renovation works of one of the railway bridges in Mechelen were finished, and a long-term monitoring program was set up. This means that the strains of the rails and bridge deck are measured under the real loading conditions.

The aim of this research is to compare the results from the real scale fatigue test with the results from the monitoring program at the actual railway bridge in Mechelen. A finite element model (FEM) is constructed to simulate the results of the real scale tests, and consequently the model is modified to imitate the real situation in Mechelen. Based on this modified FEM model, predictions can be made about the stresses induced by temperature variations and loading of the bridge or track. The data of the long-term monitoring program is used to validate whether these predictions are reliable or not.

2 REAL SCALE TESTING

2.1 Margins on All Sides

The test piece consisted of a bridge structure part on which a new steel bridge deck frame was installed. By means of fasteners, the rails are connected to the bridge deck frame. The bridge structure part originated from a demolished railway bridge in Brugge (Belgium) which had completely equal characteristics as the one in Mechelen (general arrangement, material, etc.). The new steel bridge deck frame consists of a bottom and top plate with in between longitudinal and transversal welded stiffeners. The rail type used in both the real scale test setup and the actual situation is the in Europe extensively used 60E1 or UIC60 profile. Every rail is supported by eight fasteners. A direct and elastic fastening type is used, consisting of Pandrol e-Clips and anchorage bolts. The fasteners are equipped with rubber pads to construct a type of sliding fasteners, allowing a different longitudinal movement between the bridge and the rail. A picture of the entire test piece is given in Figure 1. In five different sections of the test piece strain gauges were installed on the rails and the bridge deck, monitoring the strains in the test specimen caused by the applied loads.

![Figure 1: Test piece consisting of a part of the bridge, a new bridge deck frame, fasteners and rails.](image)

2.2 Horizontal Displacement of the Rails – Fatigue Test

The first type of test where the test piece was subjected to consisted of horizontal loading of the rails. The bridge structure was supported by two fixed bearings, one at each side of the bridge span. Relative to the neutral position, the rails were pulled and pushed over 51 mm, which meant that the rails were moving through their fasteners. The rails were loaded separately by a single piston. One rail was loaded at a rate of 2.5 mm/s while the other rail was loaded at a rate of 2 mm/s. A total of 1000 load cycles consisting of pulling and pushing the rail to a position of +51 mm and -51 mm, respectively (forward and backwards), were performed. During these load cycles, the applied load was monitored together with the strains in the rails and the bridge deck. The fasteners, which were the...
standard Pandrol type, were gradually damaged during the loading, as parts of the rubber pads were peeled and crumbled.

The horizontal loading test was followed by a fatigue test. During this fatigue test, the test piece was vertically loaded by two actuators (i.e., one for each rail), each of them situated in the middle of the bridge span. The bridge structure was supported by a horizontally fixed bearing at one side and a movable bearing at the other side of the span. During the tests, the applied loads executed by the actuators were monitored, together with the strains in the rails and the bridge deck, the axial positions of the pistons of the actuators, the vertical deflection of the bridge girder and the vertical deformations of the bearings. First, a static preload test was performed to make sure that the actuators were working in sync. The calibration was based on static loads of 40 kN, 460 kN and 500 kN. The static preload test was followed by the dynamic fatigue test. A total of 5 million cycles with the applied load varying between 40 kN and 500 kN were performed, which meant that every rail was loaded up to 250 kN. After this dynamic fatigue test, the test was concluded by a final static test where the behavior of the test piece was verified at static loads of 40 kN, 260 kN and 300 kN. The aim of the test was to verify whether fatigue cracks would appear in the structure by means of visual inspections and non-destructive capillary tests. No fatigue cracks were found.

3 FINITE ELEMENT MODEL

Based on the results of the tests executed on the real scale test piece, a finite element model (FEM) was constructed with the software package Siemens NX 12.0. The aim of this FEM model was to reproduce the results of the real scale tests without the modelling of the bridge structure.

![Figure 2. Development in time of the different load cycles during the horizontal real scale test.](image-url)

The cross-section of the rail has been drawn as accurately as possible. Based on the technical drawings of the steel bridge deck frame, it was decided to model the rail in different sections, which were ‘glued’ together afterwards. The rail is assumed to be supported by each fastener over a length of 120 mm, while the distance between two fasteners is taken as 460 mm. Train rails are installed
in a way that they are inclined towards each other. The inclination used is typically $1/20$. Consequently, the rail is rotated over an angle of $2.86^\circ$ along its longitudinal axis (Esveld 2014).

The fasteners connecting the rail to the bridge deck were modelled as springs. To model the vertical springs, the data of the real scale fatigue test has been used. First the results of the static preload test and the final static test had been compared. Since no large difference could be observed in the load-deflection curves of both tests, it was concluded that the stiffness of the test piece (and hence the fasteners) remained constant after being subjected to a cyclic fatigue loading.

To model the horizontal behavior of the bridge deck and the fasteners, horizontal springs are modelled. The monitored data of the real scale test regarding the horizontal displacement of the rails was used. The development in time of the different load cycles is represented on Figure 2.

Since the fasteners were gradually damaged during the loading, the load necessary to get the rail into motion decreased over time. This means that the fasteners become less stiff after being subjected to several load cycles. However, the rate at which the rails are displaced ($2 - 2.5$ mm/s) is not what should be expected. Hence, it is assumed that the damage to the fasteners will be kept rather limited in normal circumstances. Therefore, the monitored data of load cycle 2 (early in the test process with fasteners which are still intact) is used to model the horizontal springs. When one only looks at the parts of the load cycle where the rail is pushed or pulled until it is set in motion, the blue graph in Figure 3 can be constructed.

![Figure 3. Original data of load cycle 2 (blue) and simplification of the horizontal spring characteristics (orange).](image)

The course of the graph can be subdivided in linear gradients and horizontally constant parts. Hence, the blue graph in Figure 3 can be simplified resulting in the orange graph by selecting some well-chosen points on the original load displacement graph. When the graph remains constant, the frictional resistance between the fastener and the rail is lost and the rail can move at a constant rate without an increase of the applied load. The orange graph is the one that is used to model the horizontal springs using spring elements. Every fastener is assumed to behave in the same manner and receives the same horizontal characteristics. An important note is that the stiffness that can be derived from the orange graph needs to be divided by the number of fasteners and the number of mesh nodes representing every fastener when defining the horizontal springs in the Siemens NX model.
4 ACTUAL BRIDGE BEHAVIOR

To judge whether the ‘sliding’ fasteners are properly working at the actual railway bridge in Mechelen, a long-term monitoring program has been set up (with a duration of about one year). The aim of this program is to monitor the additional stresses in the rails and bridge deck caused by the actual circumstances. Especially one needs to keep an eye on the stresses caused by temperature variations, bridge deck rotation caused by vertical traffic loads and horizontal breaking and acceleration forces. Different types of strain gauges and sensors are used. Like the real scale test setup, strain gauges are installed on the rails and the bridge deck. Further, dynamic strain gauges are used to detect possible train traffic during the long-term monitoring program. The temperature is monitored as well by strain gauges, and displacement sensors register the distance between the upper fiber of the bridge deck and the abutment at both sides of the bridge. The registration of the measurement data occurs at a frequency of one measurement every 2 minutes. Initially a short-term monitoring program covering the loading of the track and bridge by means of a measurement train was planned to take place before starting with the long-term monitoring program. However, due to the COVID-19 pandemic this short-term monitoring program has been postponed until next year. This means that the available monitored data at the actual railway bridge can only be used to analyze additional stresses caused by temperature variations.

Based on the data of the temperature strain gauges, the theoretically expected thermal expansion or shortening of the bridge could be compared with the actual measured data of the displacement sensors. One can calculate the thermal expansion $\Delta L$ by means of Eq. (1):

$$\Delta L = \alpha \cdot L \cdot \Delta T$$  (1)

$\alpha$ is the thermal expansion coefficient for steel ($11.65 \cdot 10^{-6} \text{ m/(m °C)}$), $L$ is the span of the actual railway bridge in Mechelen (63.8 m) and $\Delta T$ is the temperature variation relative to the reference or laying temperature. Since no details were available about at which temperature the rails were installed on the bridge in Mechelen, the temperature registered at the first measurement was taken to be the reference temperature, equal to 7.29 °C. The minimum and maximum temperature reached in the considered data set were equal to -2.99 °C and +7.12 °C, respectively. Figure 4 shows the monitored displacements (blue) and the displacements predicted (orange).

The overall trend of the predictions is like the monitored data, but the peak values of the predicted expansion are often an overestimation of the reality. This difference between the theory and the reality can be attributed to the fact that the bridge does not heat up and cool down the same way along its entire structure. The upper face of the bridge deck is more exposed to sunlight and the roller support is not perfectly frictionless. Hence, the thermal expansion is perhaps not entirely horizontal and can go along with some bending of the bridge deck.

Regarding the stresses, overall, one can state that no excessive stress peaks are appearing due to the temperature variations in the monitored data. Further, the additional stresses are constant over the entire monitored part of the railway track, being about -31.0 MPa for the maximum registered positive temperature difference $\Delta T$ of +7.12°C and about +45.8 MPa for the maximum registered negative temperature difference $\Delta T$ of -10.28°C. This is not the behavior that would be expected based on what is stated in UIC Code 774-3R (Figure 5).

Based on this code, a peak stress should be expected at the bridge/abutment transition located at the roller support of the bridge. However, according to UIC Code 774-3R (UIC 2001) the additional stresses should be verified for temperature changes of ±35 °C. The temperature changes in the considered data set were a lot smaller, but one can state that under the considered circumstances the ‘sliding’ fasteners were working properly.
5 CONCLUSIONS

One can conclude that the attempt to simulate track bridge interaction with a FEM model that does not contain a modelled bridge structure has only been partly successful. The FEM model was capable to reproduce the additional stresses due to temperature differences only at the location of the abutment, and this with limited accuracy (as the FEM model underestimates the additional stresses). Further, when making the transition from the FEM model representing the test setup to the one simulating the situation at the actual railway bridge in Mechelen a problem arose with respect to the bending stiffness EI of the bridge, even though a real scale bridge structure piece had been used during the tests. Since the bridge structure is not modelled, it is not possible to verify whether all the displacement criteria with respect to the bridge deck are fulfilled. Hence, to make reliable predictions with respect to track bridge behavior it is advised to follow the methodology described in Eurocode NBN EN 1991-2 (CEN 2001) and UIC Code 774-3R (UIC 2001).

References


