VERIFICATION OF LOCAL AND GLOBAL BRIDGE ACTION OF A TIED ARCH RAILWAY BRIDGE WITH ORTHOTROPIC DECK DURING CONSECUTIVE TEST PHASES

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Abstract. Steel tied arch bridges of reasonable dimensions cannot be put in service without verification of their as built behaviour. This is generally done during a test load after which the measured results are compared to the calculated values. This paper describes the procedure for a 125 m span steel tied arch with an orthotropic deck part of the Eastern branch of the Belgian high speed railway link between Brussels and the German border. Measurements are carried out during 2 consecutive test phases. Firstly, assessment is done prior to ballast and track installation with a number of heavy lorries. Consecutively, the procedure is repeated after track installation during the crossing of a freight train at low speed. Small base strain gauges are used for this purpose in combination with a high frequency data acquisition system capable of accurately recording fast changes in the strain results which could be overlooked otherwise. The measurements are then compared to calculated values retrieved from a finite element model of the whole bridge for the main structural results and to values retrieved from a smaller but more detailed model for the local stresses in the fatigue sensitive locations of the orthotropic deck. The results indicate that the overall behaviour of the bridge complies satisfactorily to the design, although generally slightly lower values of strain and stress are measured compared to their calculated equivalents. On the other hand a comparison of the results for the lorry and train action indicate this last as a rather favourable action for the orthotropic deck as from a fatigue point of view.
1 INTRODUCTION

During recent years a number of large steel bridges have been built for the High Speed Line in Belgium. Due to vertical clearance and maximum vertical slope restrictions, the structural depth of these bridges has to be the smallest possible. For this reason a tied arch with orthotropic deck is chosen in several occasions. This combination results in a slender deck with low structural depth. The instrumented bridge is of this type and consists of a double arch with orthotropic deck serving as a tie (figure 1). Between deck and arch, diagonal hangers are installed. This structural solution generates an very high structural stiffness. This had to be the case since the loading conditions are very eccentric, due to the track curvature. This rigidity of the structure necessitates the use of extremely accurate measurement equipment for deformation measurements. Even then the reliability of such measurements is doubtful. They merely provide an order of magnitude.

More reliable measurements can be generated using small base strain gauges. They can be easily installed on steel surfaces and give accurate and reproducible values. There effectiveness remains even on the construction site and they can be protected from the environment. Measurements can be repeated, either with multiple load combinations or after the construction has been in service for some time.

The aim of the measurements carried out on the bridge under consideration was to:
- describe the general stress situation in the main load carrying elements due to a known load
- verification of assumptions and calculation models for both global behavior and local orthotropic detailing.
- determination of the construction’s fatigue resistance

For these purposes, measurements were carried out during 2 consecutive test phases prior to and after ballasting of the track. Thus generating 2 very different load schemes being a moderate, but highly concentrated load prior to ballasting by lorries and a quite larger but less
concentrating loading by freight train after ballasting.

2 TEST SETUP AND MEASUREMENT EQUIPMENT

For both global and local measurements a total of 122 strain gauges of different grid length and configuration were installed. The criteria for choosing the grid type are stress gradient, knowledge of strain and stress tensor and applicability. These considerations have led to a choice of grid length varying from 3 to 6 mm. When using these relatively small gauges, laboratory preparation is of utmost importance for a smooth application on site. This concerns the preparation of the gauges themselves as well as the preparation of the cables.

Of these 122 gauges, 40 will measure the tied arch action, whereas the remaining 82 will measure various aspects of the orthotropic plate action. As the strain gauge data acquisition system is limited to 20 gauges, 7 groups of gauges are formed. The different groups are chosen to each represent a different phenomenon:
- group 1: global tied arch action – south arch
- group 2: global tied arch action – north arch
- group 3: orthotropic deck: longitudinal stiffeners
- group 4: orthotropic deck: crossbeam (in plane effect)
- group 5: orthotropic deck: deckplate (midspan)
- group 6: orthotropic deck: deckplate (at crossbeam)
- group 7: orthotropic deck: crossbeam (out of plane effect)

Strain measurement is done through a 20 channel dynamic acquisition system. This system has an individual bridge completion, signal amplification and filtering per channel. Up to 10,000 samples per second per channel can be reached. This was however not necessary for these measurements where the sampling rate could be limited to 100 samples per second without loss of information. A static or quasi-static measurement would however have lead to a loss of accuracy, since the signal can change fast due to concentrated loading and narrow influence lines. Figures 2 and 3 give an impression of the strain gauges and the measuring cables.

Figures 2 and 3: Impression of strain gauges and measuring cables.
The first phase of the measurements is done prior to ballasting, directly on the orthotropic deck with 16 fully loaded lorries of 44 ton, totaling 704 ton. These sixteen lorries are used only for the global bridge action. For the detailed orthotropic deck measurements, 1 truck is sufficient. Axis positions and axis loadings are however recorded individually for this truck. The second phase of the measurements is done after ballasting and track installation with a 1069 ton freight train with 2 locomotives. During measurement the train moves at relatively slow speed along its curved track across the bridge. The reader will notice that, for the global measurements, both loads represent only a fraction of the bridges design load, being over 2000 ton.

3 GLOBAL TIED ARCH PERFORMANCE : PHASE 1 AND 2

3.1 Phase 1

During this first phase, 16 lorries are positioned on the bridge in 3 different load positions, namely a full load scheme and 2 chessboard loadings. Strains are recorded on the arch, on different hangers, on the main lateral tie girders and on the lower side of the deckplate. The recorded strains are than transformed to stress results and are compared to calculated values at the corresponding locations of the finite element model presented in figure 4.

As already mentioned in paragraph 2, the applied load in this first phase reaches only 35% of the actual design load. Thus, recorded strains and stresses are relatively small. The size of the strains inevitably creates generally smaller accuracies. In this case, this is however not necessarily so. Although the recorded values are quite consistent with calculated values (figures 5 to 7), the latter are almost systematically higher. The mean value of the ratio of recorded to calculated values is 78%. These differences are to be attributed to an over estimate in the calculated values rather then due to inaccuracy of the measurements. In addition, the difference is somewhat larger for the transversely unequal chessboard loadcases, as for the full load. However in previous testcases for comparable bridges equal ratio’s were
obtained.

Figure 5: Comparison of calculated and recorded values: Fully loaded bridge

Figure 6: Comparison of calculated and recorded values: Chessboard load 1.

Figure 7: Comparison of calculated and recorded values: Chessboard load 2.
3.2 Phase 2

During this second phase, a 1069 ton freight train with 2 locomotives has crossed the bridge several times at moderate speed. Although this train has a larger total weight then the lorries, in fact it generates an even smaller load on the bridge as its length exceeds the length of the bridge. As the train crosses however on one track, the load is more eccentric as compared to the lorry load. The maximum recorded strain and stress values are therefore in the same order of magnitude as those generated in phase 1.

Due to the fact that quite some time passed between both phases, the general condition of the strain gauges was worse resulting in some signal interference. This is clearly visible in figure 8 comparing recorded and calculated values for the stresses on both arches.

The results given in figure 8 give values in the arch about 2 m from the support. Stresses are measured at the 4 corners of the arch cross section. As the arch is a box girder, these 4 points provide a full view of the horizontal and vertical stress distribution in the arch cross section under consideration. Figure 8 indicates that, although generally calculated and measured results coincide, the upper side measured values are smaller in absolute value as compared to their calculated equivalents. This may be due to the fact that the base of the arch has a very complex shape, compared to the calculation model. In addition to this, we observe a number of smaller stress cycles due to the crossing of each axle or bogie. The magnitude of these cycles is however small and will not cause any fatigue damage. Again it is clear that all measured and calculated values are small (maximum 25 MPa), due to the fact that the design load exceeds the test loads.

![Figure 8: Phase 2: Comparison of calculated and measured results.](image-url)
4 ORTHOTROPIC DECK PERFORMANCE : PHASE 1 AND 2

4.1 Phase 1

Prior to ballasting and track installation the orthotropic deck behaviour was assessed using a fully loaded 5 axle lorry. During measurement the vehicle drives at relatively slow speed along predetermined lines parallel to the bridge axis. The transverse positions of the lines on which this vehicle was moving were chosen to derive a kind of influence line for transverse load positions since the effect of minor changes of the load position changes the stresses at the locations under consideration. Three locations are chosen at 975 mm, 1190 mm en 1405 mm from the edge of the deck. In addition, the geometry of the vehicle itself consists of 2 different types of wheels. Whereas the truck’s first and the 3 trailer axles are 2 wheel axles, the truck’s second axle is a 4 wheel axle.

From a fatigue point of view an important location in this deck is the connection between the longitudinal trapezoidal stiffeners and the deckplate. When using closed trapezoidal stiffeners the welded connection is one only one sided. This results in large stress concentrations, especially at the weld root. When this weld is executed as a fillet weld, the fatigue performance is extremely poor. When it is however executed as a penetration weld, the behaviour is largely improved, although still doubted in coded values.ii

Of the three transverse positions, the second proves to give the largest values, which can be seen in figure 9. The crossing at 1190 mm introduces a transverse stress cycle at 40 MPa, a stress cycle at 105 MPa and 3 stress cycles at 80 MPa. These last 4 stress cycles can cause extensive fatigue damage at this location when compared to the Eurocode fatigue criterion. As can be seen from the graph, all these stress cycles have a compressive mean. However, as this is a welded detail, this compressive mean should not affect the fatigue endurance of the detail.

The maximum values are compared to results from finite element calculations. This finite element model consists of 2 fields of 12 trapezoidal stiffeners supported longitudinally by the main beams and transversely by the crossbeams (figure 10).
The maximum values are compared to results from finite element calculations. This finite element model consists of 2 fields of 12 trapezoidal stiffeners supported longitudinally by the main beams and transversely by the crossbeams (figure 10). The Mindlin elements used in the model generate upper and lower side stress results, providing the necessary components for verification of the longitudinal and transverse strains at the lower side of the deckplate at the location under consideration.

Figure 10 : Finite Element model of the orthotropic deck.

Being most representative in terms of maximum stress values, the results are given for the transverse load position 1190 in figures 11 to 13. The left hand side figures display the area between the first at the second longitudinal stiffener, at midspan between crossbeams as is indicated in figure 11. The right hand side figures show transverse cut at that same location proving very good accordance between calculated and measured results. In the graphs the measured values are indicated as dots. Phase one has proven the existence of large transverse bending stresses in the deckplate due to the high local tyre loads. Should this configuration be used for a road bridge with a shallow surfacing, these stress cycles are unacceptable and would lead to premature fatigue failure. Of course, since this is a railway bridge, loads are fundamentally different. The concentrated axle loads are distributed through the system of rails, sleepers and ballast providing a load dispersal and reducing the stresses at the stiffener to deckplate intersection considerably. This effect has been investigated in phase 2.

Figure 11 : Truck’s first axle : confrontation of finite element calculations and measurements
4.2 Phase 2

After ballasting and track installation, but before effectively being in operation for the high speed line, the behaviour of the orthotropic bridge was assessed using a 1069 ton freight train with 2 locomotives. During measurement the train moves at relatively slow speed along its curved track across the bridge. Of course, for the assessment of stiffener to deckplate fatigue, a train of this length is unnecessary. A simple locomotive would have been sufficient for this purpose. However at the time of measurements, other effects were monitored as well.

Since the global effect of the freight train is quite considerable, compared to the effect of one simple truck in the first testing phase, stresses are generated in the deckplate caused by 3 different collaborating effects. Firstly the deckplate serves as the effective plate between the stiffener webs, producing predominantly transverse stresses. This is the action being considered. In addition, the plate also serves as top flange of both longitudinal and transverse stiffeners, giving positive and negative stress results due to the effective load carrying activity of the orthotropic deck itself. This action creates predominantly longitudinal stresses. Finally, the orthotropic plate forms, together with the edge member the tie chord of the arch. This creates in all parts of the orthotropic deck a longitudinal tensile stress state.
In figure 14 the recorded values due to the crossing of the two locomotives only are given. The graph is a filtered result from the initial graph, including the full freight train. The stresses shown in figure 14 are found at the same location as those shown in figure 9.

![Figure 14: Recorded Stress values at stiffener to deckplate intersection](image)

Clearly, although the axle load is much higher than the one generated by a truck, the measured stresses are much lower. This is due to 3 separate actions: First, due to the stiffness of the rail, the load is actually carried by more than one sleeper. This reduced the patch load to half its original value. Second, due to the ballast the load is dispersed to a much larger area, roughly twice the sleeper area, depending on its thickness. And third, due to the ballast stiffness itself, and the flexibility of the plate area between the stiffeners web, the load is attracted to these webs, creating much smaller bending moments in the plate.

5 CONCLUSIONS

In this paper the results of measurements on a tied arch during 2 consecutive measurement phases are presented. The results are than compared to those of finite element calculations. Concerning the global measurements, the comparison of results is favourable, although differences are found at the arch supports, possibly due to the modelling of this complicated connection. Concerning the local measurements, the comparison of results is good, revealing indeed very large transverse bending stresses in the deckplate due to concentrated wheel loads. After ballasting and track installation, these stresses are reduced to negligible values due to the load dispersal through rails, sleepers and ballast.

REFERENCES

[ii] De Corte W., An improved detail category for trapezoidal stiffener to deckplate welds, based on full scale stress data and fatigue tests, Proc 3rd Int Conference on New Dimensions in Bridges, Kuala Lumpur, Malaysia April 2003 pp. 139-149