ACCURACY OF LOAD TEST ON A DOUBLE-CURVED CLOSED SECTION VIADUCT

Philippe Van Bogaert
Civil Engineering Dept, Ghent University,
Belgium

This paper presents the results of non-destructive load testing of a double-curved steel railway viaduct. Its objective is to demonstrate the accuracy of the test method and of the FE-model used for the detailed design of the bridge. Due to the double curvature of the structure being considered, the Zemst bridge shows particular characteristics, which cause unequal stress distribution, especially in the wide lower flange. In spite of the horizontal curvature of the structure, no significant differences in the deformations of both edge members were found during placing of the bridge. Deformations seemed to correspond better to a simple beam model than to the complicated FE-model. This is not true for the stresses, measured during the load test. The effect of unequal stress distribution in the lower flange is clearly found at the middle span centre, but not systematically in other measurement sections. Comparison of all relevant measurements to results from the FE-model shows that the ratio of measured to calculated values has an average value of 80%, whereas the 95 % fractal value of this ratio is close to 1. Although the numerical model renders conservative values, the agreement with measured values is satisfactory. The results from stress measurements demonstrate that the initial aim to use 157 strain gauges was ambitious. Only 26 measurement locations gave relevant data. Obviously, low measured stresses may also be relevant since they demonstrate the absence of possible stress concentrations. However, measurements should have a well defined purpose and the number of locations should be limited.

Corresponding author's email: philippe.vanbogaert@Ugent.be

ACCURACY OF LOAD TEST ON A DOUBLE-CURVED CLOSED SECTION VIADUCT

Philippe Van Bogaert[1]

[1] Civil Engineering Department, Ghent University, Belgium

ABSTRACT: This paper presents the results of non-destructive load testing of a double-curved steel railway viaduct. Its objective is to demonstrate the accuracy of the test method and of the FE-model used for the detailed design of the bridge. Due to the double curvature of the structure being considered, the Zemst bridge shows particular characteristics, which cause unequal stress distribution, especially in the wide lower flange. In spite of the horizontal curvature of the structure, no significant differences in the deformations of both edge members were found during placing of the bridge. Deformations seemed to correspond better to a simple beam model than to the complicated FE-model. This is not true for the stresses, measured during the load test. The effect of unequal stress distribution in the lower flange is clearly found at the middle span centre, but not systematically in other measurement sections. Comparison of all relevant measurements to results from the FE-model shows that the ratio of measured to calculated values has an average value of 80%, whereas the 95 % fractal value of this ratio is close to 1. Although the numerical model renders conservative values, the agreement with measured values is satisfactory. The results from stress measurements demonstrate that the initial aim to use 157 strain gauges was ambitious. Only 26 measurement locations gave relevant data. Obviously, low measured stresses may also be relevant since they demonstrate the absence of possible stress concentrations. However, measurements should have a well defined purpose and the number of locations should be limited

1 INTRODUCTION : THE ZEMST RAILWAY OVERPASS

The double curved steel railway bridge at Zemst allows the crossing of the new railway line from Brussels to Mechelen above half of the E 19 motorway between Brussels and Antwerp. It is part of the Diabolo project, which consists of improving the road and railway connections with Brussels' Airport. The main part of the new railway connection runs in the central reservation of the motorway and the crossing at Zemst, allows leaving this location and joining the railway station of Mechelen, where additional tracks will be built.

The geometry of the crossing is shown in fig. 1. The double-track shows a bend on 1282 m, as well as a vertical rounding of 6000 m. These data allow for maximum train speed of 160 km/h. However, the crossing with the subjacent motorway is very skew, since the mean angle equals 13.7°. Because of this, the structure consists of a three-span

continuous steel closed section girder, with single piers, adjacent to the motorway edge. This is discussed more in detail in Van Bogaert (2010), including the requirements of the loading schemes and acceptable deformations. The aim of the design was also to limit the height of the structure, in order to avoid motorists to be distracted from the motorway. In addition, the bend of the subjacent tracks gave the opportunity for building a smoothly variation of the structure's alignment, which resulted in a double curve, both of the cross-section and of the bridge axis. Eventually, this has lead to the cross-section of fig. 2, which as an apparently closed-section, with lateral stiffening beams of bent steel plates. It might be noticed that the cross-section is closed only by the concrete bridge deck slab, which is connected to diaphragms. These diaphragms continue inside the lateral members, whose cross-section consists of a series of circular segments. The height of the steel crossbeams is limited to 1.45 m plus 0.24 m of concrete slab. Higher torsion stiffness is found in the lateral members and less in the central part of the section, partly because of the curved shape and partly by the diaphragm stiffeners. The curvature of the lateral members does not contribute highly to the torsion resistance. However, it clearly demonstrates how the members are acting and adds aesthetical value to this longitudinally and transversely curved structure.

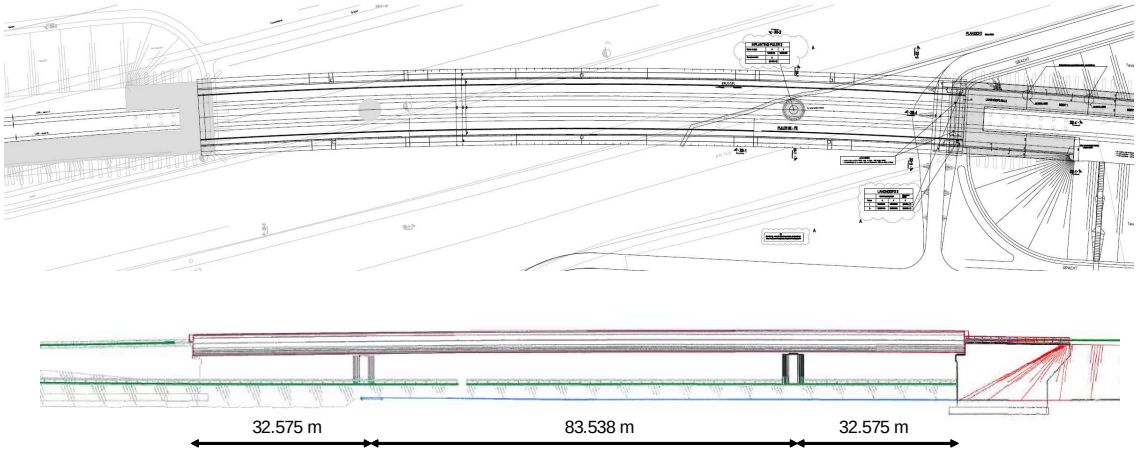

Figure 1. Plan and front view of Zemst overpass.

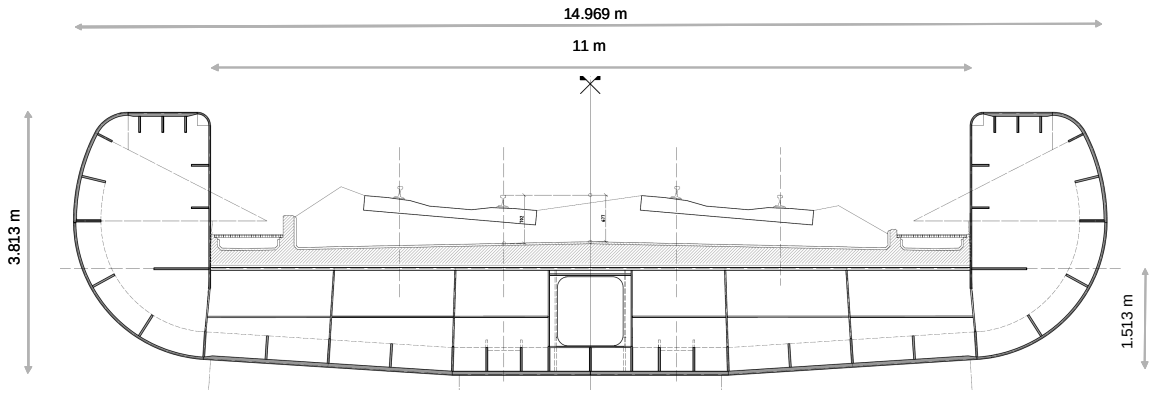

Figure 2. Regular cross-section of Zemst overpass.

Evidently the superstructure of this bridge shows some particular characteristics and the basic analysis was carried out by means of an elaborate FE-model. The aim of the load test was to verify the accuracy of the FE-model and the actual behaviour of the structure, in particular in comparison with simple beam theory.

2 DEFORMATION MEASUREMENTS

During the assembling of the steel structure, which was welded on the building site from 21 parts, prepared in a steel workshop, and the subsequent placing in its final position, deformations of the structure have been monitored. This allowed assessing the effect of the dead weight of the steel structure, since the latter was welded according to precalculated rises and subsequently handled with hydraulic moving lifting devices to its final position. This can be seen in fig. 3. Actually, during lifting, the structure was supported at 8 points, with equal reaction force, as the pressure in each hydraulic jack is kept equal.

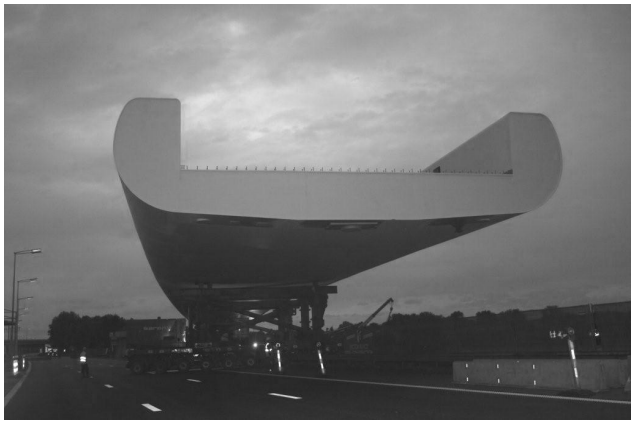

Figure 3. Overnight lifting and transportation of steel structure.

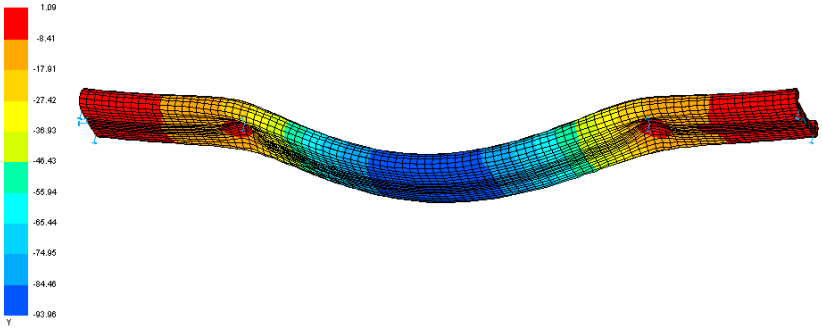

Figure 4. Vertical deformations due to dead weight, determined by FE-model.

After placing, the reactions at the ends and piers become active and the deformations due to dead weight as calculated through the FE-model are seen in fig. 4. A maximum sag of the central span of 93.95 mm can be observed. The actual deformations of both edge members can be compared to the calculated ones as well as to values obtained by a simple beam model. This allows to validate the overall accuracy of the FE-model, as well as to assess the different behaviour of both edge members, due to the horizontal curvature of the structure. In addition the similitude with the simple beam model has been tested.

The results of the comparison are shown in the chart of fig. 5. Apart from the local effect in the left approach span, there is no significant difference between the inner and outer edge member deformations. This demonstrates that the horizontal curvature has little effect. In addition, the structure seems to be stiffer than calculated by the FE-model, the ratio of measured deformations being 80% of the predicted value, or 47.63 mm for the inner member and 55.31 mm for the outer member. The reason for this is the

larger transverse stiffness of the crossbeams above the piers, which show considerable larger stiffness than expected, as well as the overall dimensions of the bearings at this location, by thus spreading the reaction at the pier supports.

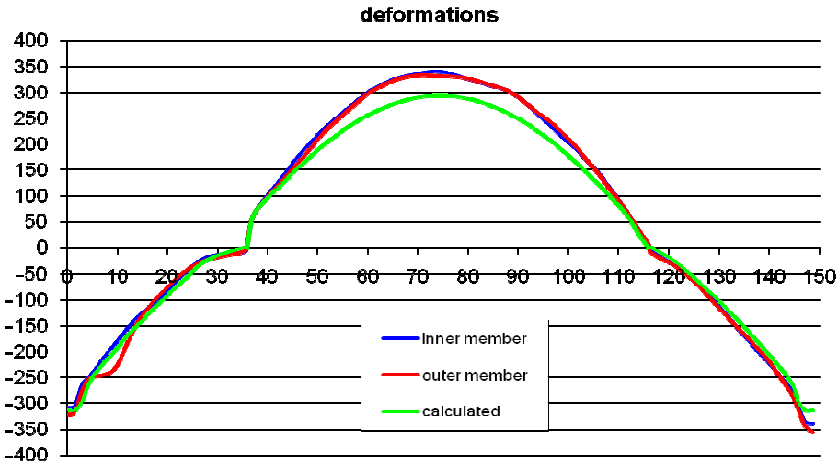

Figure 5. Measured vertical deformations.

The simple beam model results in vertical deflection of 55.42 mm instead of 93.95 mm. If corrected the crossbeam deflection, the FE-result would be 68.95 mm, which is still larger than the results of the beam model. Hence, concerning deformations, the simple beam model is sufficiently accurate to simulate the structure being discussed

3 STRESS MEASUREMENTS

The structure was equipped with 157 strain gauges in 5 cross-sections during the test load. Because of the danger of working above the motorway, all gauges were glued inside the steel box, which allowed easy access and walking from one compartment to the next, through the holes in the diaphragm stiffeners. The large number of gauges was due to uncertainty of the stress distributions across the wide lower flange.

Figure 6. Lorries placed on bridge during load test.

A total of 24 heavy lorries were used as loads during testing. Each lorry has a mass of approximately 45 tons and the total load equals 10800 kN. The lorries were placed in various positions, also at maximum eccentricity with respect to the bridge axis. The lorries, placed on the bridge can be seen in fig. 6. A double line of lorries was placed across the whole length of the bridge, which obviously does not correspond to the critical loading condition. However time failed to actually implement all load positions

as imagined previously. Recording of strains has been guaranteed with accuracy of 1μStrain as has been proven on many other cases. The uncertainty of stress distribution across the steel cross-section is shown by fig. 7 on which the normal stresses are displayed by means of bars of representative length. In this figure, the full loading case, with symmetrical position of the lorries is shown. Obviously, the right hand upper flange shows an abnormal result and the measurement is considered as inaccurate. All other results are acceptable and are being considered for further comparison. The results also demonstrate that the edge members are acting as stiffening beams, since the lower flange stresses are distinctly higher than at the lower plate centre. This is demonstrated by comparing with stresses from a simple beam model, summarized in table 1.

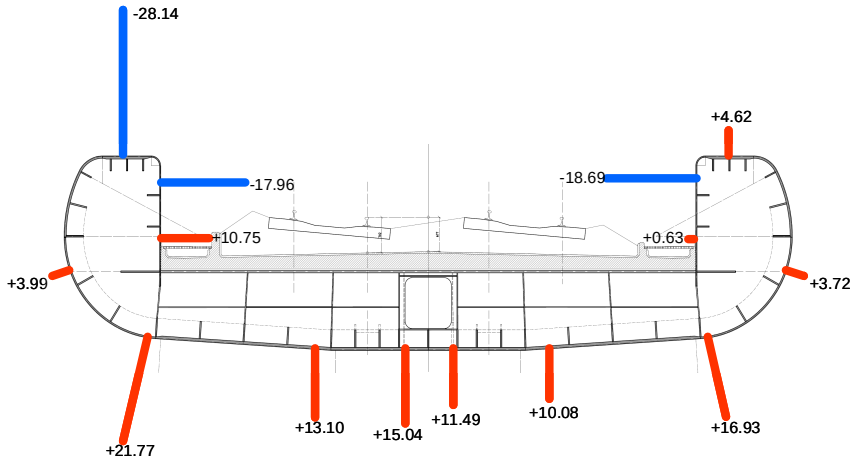

Figure 7. Distribution of measured stresses across section for position 1.

Table 1 : comparison of beam and FE stresses with measured values

	Midspan centre		Pier section	
	$\sigma_{up,min}$	$\sigma_{low,max}$	$\sigma_{up,max}$	$\sigma_{low,min}$
Measured	-28.14	21.78	23.42	-18.48
Beam model	-26.18	20.70	28.41	-12.98
FE-model	-34.80	20.22	36.76	-26.68

The numerical values of table 1 indicate that the beam model can be accurate, but certainly fails at the pier section lower flange. The results from the FE-model overestimate most measured values and is certainly conservative. Hence, a design based on a simple beam model would fail, whereas the complete FE-model is certainly reliable and the structure's adequacy is fully established.

4 VALIDATION OF RESULTS

Further validation of results may be obtained by considering the 3 loading cases, for which measured values are available. These are called positions 1, 6 and 7, those being the numbers of the original programme. For all three positions the bridge has been fully loaded, although this is not the most critical condition. Position 1 corresponds to centric loading, whereas positions 6 and 7 correspond to eccentric loading, towards the inside of the bend and towards its outside. Relevant results may now be compared to FE-predicted values. The middle span central section, in particular the upper- and lower

flanges certainly are relevant. In fig 8, block charts are shown for the upper- and lower flange in this section for the 3 positions of lorries. The agreement is distinctly good, especially since the scale of the vertical axis is truncated.

In this section, the stresses in the edge member vertical web were also measured. In spite of their discontinuity and lack of connection to the lower plate, these vertical webs contribute actively to the section resistance, as can be seen from fig. 7. This is an important characteristic, since it demonstrates that the type of cross-section being implemented actually performs well. The comparison of measured and FE-calculated values is shown in fig. 8. Except for position 7, again the agreement is certainly good and this type of measurement is relevant

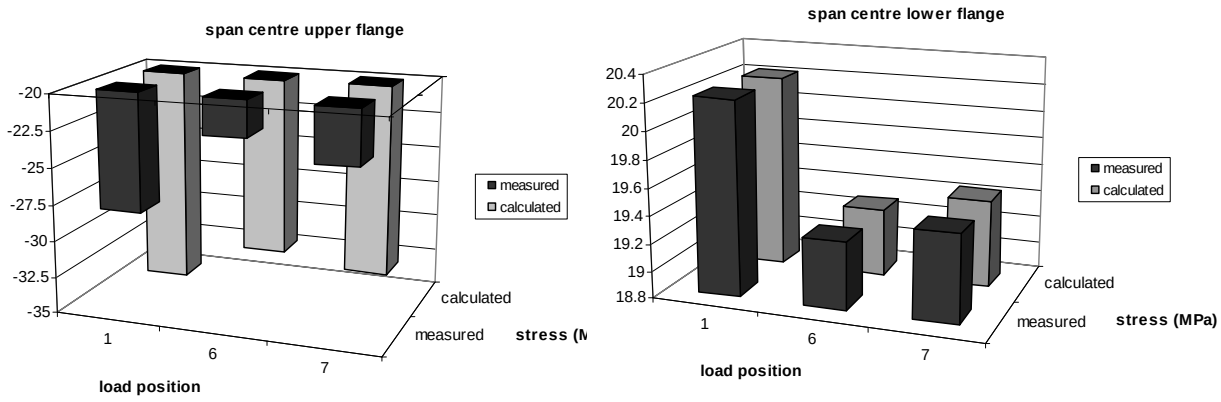

Figure 8. Measured and calculated (FE) values of stresses in span centre.

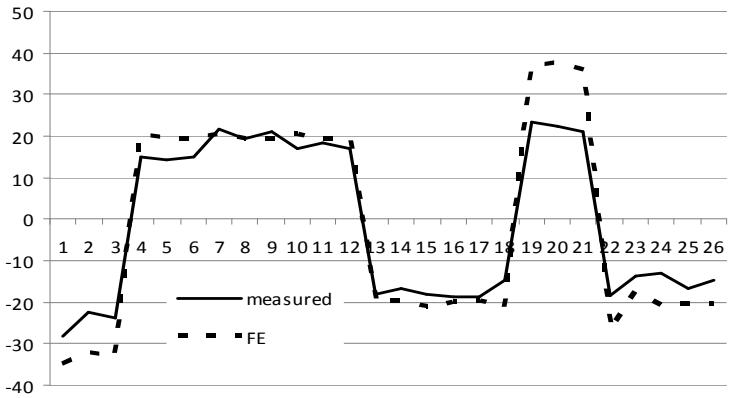

Figure 9. Overall data of stresses larger than 15 MPa..

Other sections show low stresses and as the reliability of measurements does not exceed 1 μStrain, the results can only be compared with maximum values. Because of this, the stress distribution across the lower flange does not always show the effect as displayed in fig. 7. However, unequal distribution can be found, although an edge beam effect is not systematical. The ratio of maximum lower flange stress to the value at the central point varies from 1.45 to 1.69. As mentioned before, this is not entirely confirmed by the FE-modal, although all measured stresses are lower than calculated values. For instance, table 2 shows results for the upper- and lower flange for a section at the quarter of the central span. Again good agreement is found. Summarizing all sufficiently high stress results, the graph of fig. 9 has been drawn, comprising 26 relevant results, this number being sufficiently high to calculate average and standard

deviation values. The graph comprises values from the central- and pier as well as from quarter span sections, from various locations in each section and of course from both signs. The only criterion to take a location into account, has been that the absolute value should at least equal 15 MPa. Thus statistical values can be determined with sufficient reliability. The result can be seen in table 3, which displays an average value of the ratio of measured value to FE-calculated one of 80%. The standard deviation being 17% of the average, the 95% reliability value of the ratio equals 1.027

Table 2 : comparison of measured and FE stresses at quarter span

	upper flange		lower flange	
	$\sigma_{up,min}$	$\sigma_{low,max}$	$\sigma_{up,max}$	$\sigma_{low,min}$
Measured	-16.653	-14.742	6.321	5.481
FE-model	-20.47	-20.47	6.4	6.4
ratio	1.23	1.39	1.01	1.17

5 CONCLUSIONS

The results from stress measurements demonstrate that the initial aim to use 157 strain gauges was ambitious. Only 26 measurement locations gave relevant data. Obviously, low measured stresses may also be relevant since they demonstrate the absence of possible stress concentrations. However, measurements should have a well defined purpose and the number of locations should be limited.

Nevertheless, comparison of all relevant measurements to results from the FE-model shows that the ratio of measured to calculated values has an average value of 80%, whereas the 95% fractal value of the ratio is close to 1. This result demonstrates that the testing procedure may be considered as reliable and the FE-model accurately simulates the behaviour of this unusual structure, although the latter renders conservative values. .

REFERENCES

Van Bogaert, Ph. Design and Construction of a double-curved railway overpass and 3-track tubular arch. Report IABSE-Symposium Large Structures and Infrastructures for Environmentally Constrained and Urbanised Areas. Venice Sept 22-24 2010. pp 564-565 + 8pp CD-Rom ISBN 978-3-85748-122-2