Investigation of the historical Osobowicki Bridge in Wroclaw

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ABSTRACT: The Osobowicki Bridge was constructed in the last decade of the 19th century as one of the most important bridges over the Odra River in Wroclaw (Poland). As a background for the reconstruction planning, including historical value of the bridge and its aesthetical importance, the investigation of the structure has been performed by the Wroclaw University of Technology. The paper presents the programme, execution and results of the research study consisting of three main parts: detailed inspection of the structure condition, experimental field tests of the bridge and theoretical analysis of the masonry structure based on the advanced FEM model. The experimental and theoretical results are compared and discussed to formulate the general evaluation of the bridge condition and required improvements of the structure. Special attention is paid to the proposed rehabilitation works based on the results of tests and analyses.

1 INTRODUCTION

In 2009 on the basis of agreement between the road authority of Wrocław City and Wrocław University of Technology a detailed investigation of the Osobowicki Bridge was performed. The reason for the task was a need to increase the permissible service load on the bridge located along one of the main national roads crossing the city. Other objectives were evaluation of the technical condition of the bridge structure and determination of the required maintenance and repair works.

Within the scope of works, there were several inspections and tests carried out at the bridge. The most important tasks were:

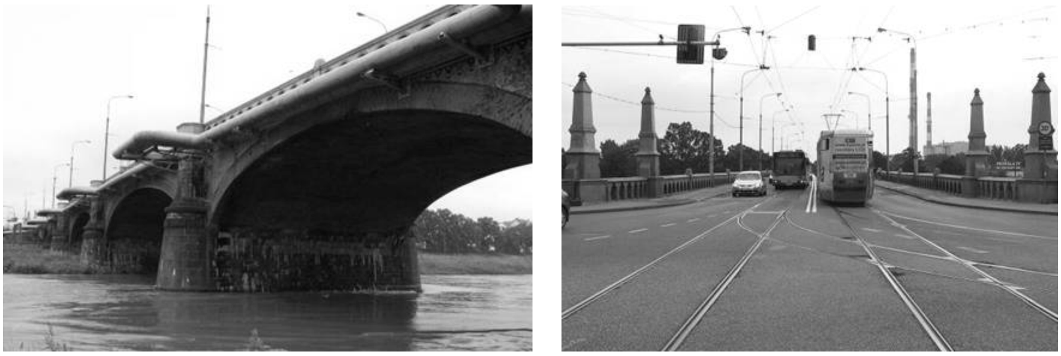
(1) Search and study of historical documentation of the bridge,
(2) Visual inspection and photographical documentation of the bridge technical condition,
(3) Precise measurements of the bridge geometry,
(4) Thermographical inspection of the structural parts and of the pipeline system,
(5) Measurement of the static and dynamic bridge response to the traffic loading,
(6) Numerical analysis of the bridge including comparison with the results of field tests,
(7) Assessment of the technical condition and the load carrying capacity of the bridge taking into account defects of the structure.

2 DESCRIPTION OF THE BRIDGE

The Osobowicki Bridge is located in Wroclaw in the south-west of Poland and crosses the Odra River. It was erected in 1895-97 during a comprehensive reconstruction of the city river system. The structure survived war periods as well as an intensive flood in 1997 without any serious defects. In the second half of the 20th century there were large pipelines with their supporting structures installed along both sides of spans.

Currently the bridge conducts one of the main national roads, tram and pedestrian traffic. The general view of the structure is presented in Fig.1 and Fig.2.

Figure 1 : The general view of the Osobowicki Bridge in Wrocław.



(a) (b)

Figure 2 : Osobowicki Bridge: (a) main spans with visible pipeline system, (b) bridge deck with the usual traffic.

The structure of the bridge is composed of 8 three-centred arches of spans of: 21,10 m + + 24,20 m + 28,30 m + 32,30 m + 28,30m + 24,20 m + 21,10 m + 18,00 m. The total length of the bridge is 242,80 m. The geometry of the bridge is shown in Fig.3.

Width of spans is 16,00 m and thickness of masonry arch barrels is variable for various spans (crown thickness from 77 up to 90 cm). Supports of the bridge are made of concrete with stone facing and spans (including arch barrels and spandrel walls) are made of bricks.

The maximum admissible mass of vehicles crossing the bridge is limited to 30 tonnes.

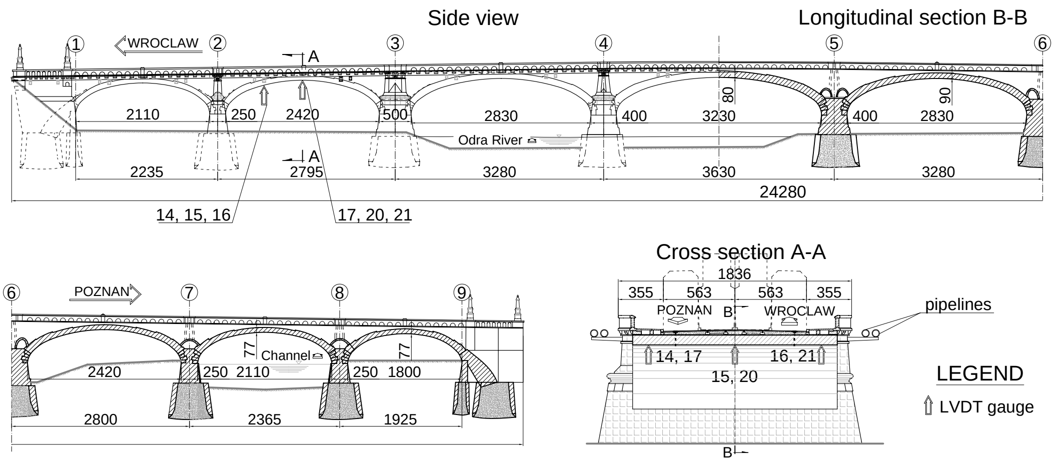


Figure 3 : Geometry of the bridge: side view with longitudinal and cross sections.

3 TECHNICAL CONDITION OF THE BRIDGE

Detailed survey of the technical condition of the bridge including visual and thermographical inspection was performed by Bridge Group of the Wroclaw University of Technology in summer 2009. Within the scope of the survey all components of the bridge were investigated.

(a) (b)

Figure 4 : Typical defects: (a) longitudinal cracking and moistening of arch barrel: (b) leaching at arch barrel springing.

(a) (b)

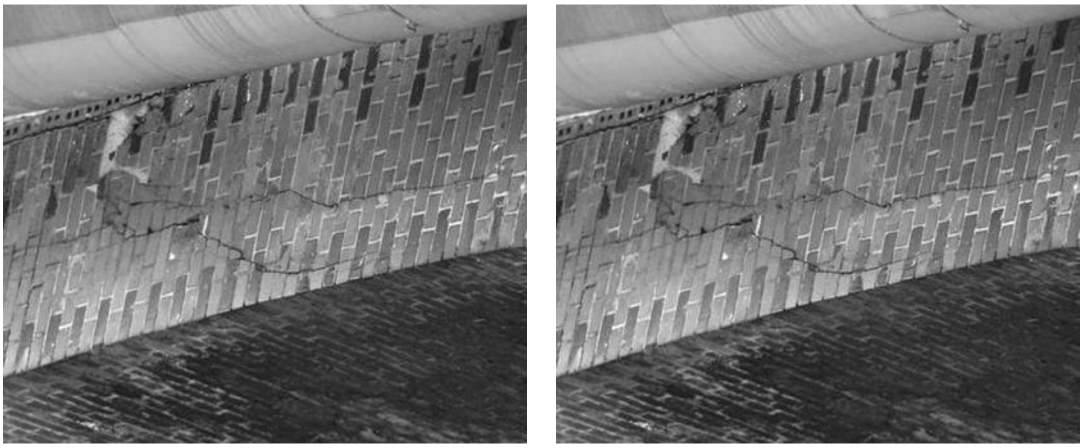
Figure 5 : Typical defects: (a) longitudinal cracking along the edge of arch barrel: (b) losses on the pavement surface.

The overall condition of the structure was estimated as satisfactory however several worrying but mainly typical defects were discovered. The identified defects and the degradation processes were categorized according to classification system proposed by Bień and Kamiński.

The main deficiencies of the structure are related to inefficient waterproofing of spans which causes moistening and in consequence leads to destruction of the masonry material, its cracking and losses (see Fig.4). Along the edges of most of the arches there are present longitudinal fractures separating these parts of arch barrels from their central parts (Fig.5a). This effect is most probably influenced by large pipelines attached to spandrel walls (visible in Fig.2a).

At the fronts of the arches there are also several areas of cracking going through the arch or between the arch barrel and spandrel wall (Fig.6).

For evaluation of some local inhomogeneities of the structural material as well as for detection of moistening concentrations, there was a careful thermographical inspection performed. Except for some local increase of moisture in the vicinity of the drainage pipes in arch barrels no other special areas of moistening were discovered. It could however be caused by dry period before and during the test.

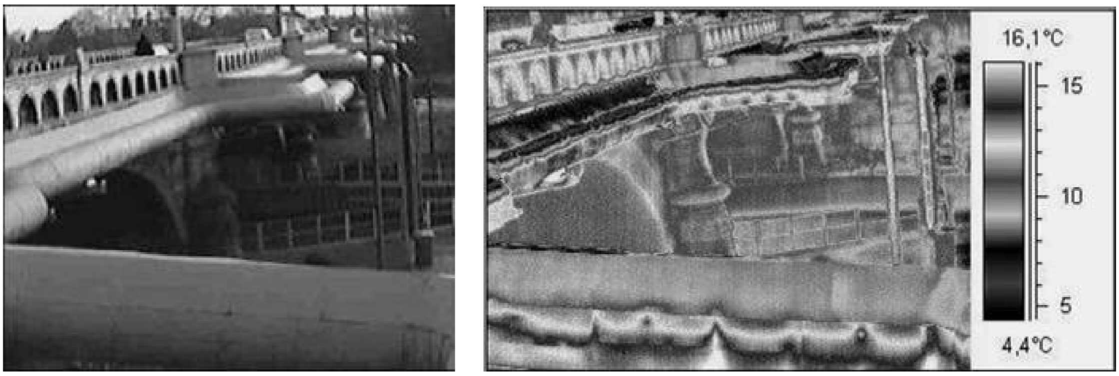


(a) (b)

Figure 6 : Typical defects: (a) Skew cracking of the arch front, (b) fracture between arch and spandrel wall.

The thermographical inspection was also applied in survey of the pipelines attached to the bridge. During this test an extensive damage of the pipelines' thermal cover and its dysfunction were confirmed (see Fig.7).

Generally on the basis of the acquired observations very poor condition of relatively "young" bridge equipment against quite good condition of the "old" structural components was found. Serious damage of the pavement surface on the bridge is also visible (see Fig.5b).



(a) (b)

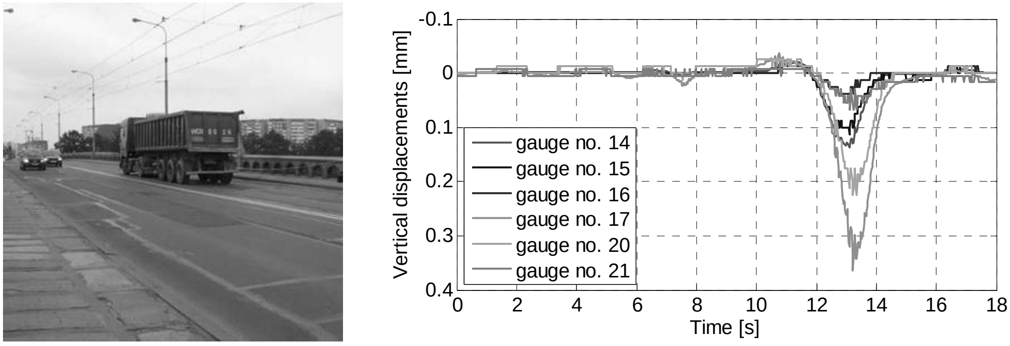
Figure 7 : Inspection of spans 6-7 and 7-8: (a) photographic image, (b) thermographical image.

4 TESTING UNDER TRAFFIC LOADS

One of the important elements of the bridge assessment were tests under the traffic load. Scope of the tests was aimed at evaluation of static and dynamic behaviour of the structure under the usual traffic including passenger and freight motor vehicles as well as trams.

Response of the structure to the traffic load was measured by means of 8 LVDT gauges and 12 accelerometers located on spans 1-2 and 2-3. Six LVDT gauges were applied to measurement of vertical displacements of arch barrel in the middle and in quarter of span 2-3 (Fig.3). Next 2 LVDT gauges were installed in the middle of span 1-2 to measure relative horizontal and vertical displacements of two parts of the span separated by longitudinal cracking (visible in Fig.4a) in the middle of the arch width. The accelerometers were located at the deck along both edges of the superstructure, in the middle and in quarters of spans no. 1-2 and 2-3. These gauges were inertial accelerometers measuring vertical accelerations of the structure.

During the tests several tens of random traffic runs were controlled among which some representative loading schemes were selected. One of the typical loading schemes giving the highest values of the span deformation is presented in Fig. 8a. It is represented by a single truck with 3-axle rear bogie with about 100 kN per axle crossing the bridge along the right lane. The biggest displacement caused by such loading was measured in the middle of the span at the edge and reached 0.35 mm. Results of displacements' change in time during passing of the representative truck for all six monitored points of span no. 2-3 are presented in Fig.8b.



(a) (b)

Figure 8 : Representative traffic loading: (a) photographic view, (b) results of displacements.

As aforementioned two LVDT gauges were applied to monitor the behaviour of the arch no. 1-2 in the area of cracking. Result of this control indicated no horizontal nor any vertical relative displacements of the arch parts.

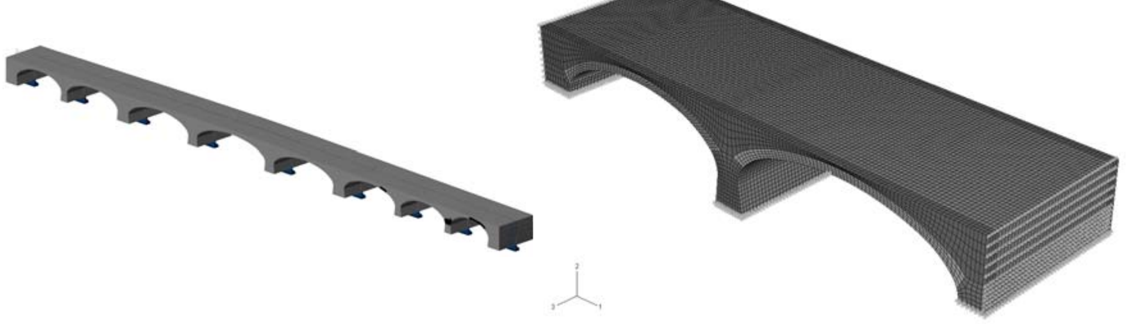
Simultaneously with measurement of displacements also vertical accelerations of spans no. 1-2 and 2-3 were controlled to evaluate dynamic sensitivity of the structure. Received results were applied to determination of the dynamic amplification factor (DAF) as well as to modal analysis of the spans. The maximum measured accelerations did not exceed 0.2 m/s^2 and experimentally determined DAF was 1.17 which occurred to be higher than its theoretical value equal 1.05 calculated according to the Polish code (PN-85/S-10030). By means of modal analysis the basic mode shapes were identified; the first flexural mode was found at frequency of 6 Hz. Expected high dumping of the structure was confirmed.

5 STRUCTURAL ANALYSIS OF THE BRIDGE

One of the most important aims of the presented condition assessment was evaluation of the load carrying capacity of the structure. The approach applied to the task was an analysis performed by means of the Finite Element Method. Two types of numerical models were utilized: simplified 2D model of the whole structure (composed of 9094 quadrilaterals) and advanced 3D model precisely representing two spans of the bridge (built of 55678 bricks). The models are shown in Fig.9. The aim of the analysis was twofold: it was applied for evaluation of the behaviour of the structure under a typical service load as well as for assessment of the load carrying capacity taking into account code loads.

In the first step, the analysis was aimed at calibration of the applied numerical models by means of the test results presented in the previous chapter. Similar methodology was proposed by Bień et al. The calibration process concentrated on selection of basic material parameters' values providing the most compatible output of calculations with the results of measurements. Load scheme applied for comparison was a typical one noticed during the test and giving the highest displacements – a 300 kN truck boogie mentioned before.

Finally, the accepted basic material parameters (unit weight and modulus of elasticity) providing the most satisfying results are shown in Table 1. Values of other material parameters were assumed as typical ones according to findings presented by Bień and Kamiński. An intense orthotropy of the masonry arch barrels is revealed: it has much higher stiffness in the transverse than in longitudinal direction. This feature is characteristic for masonry structures constructed with mortar bed joints limiting their stiffness in perpendicular direction to the joints (see Kamiński 2007). The other parts of the bridge were modelled as isotropic materials.



(a) (b)

Figure 9 : Numerical models applied in analysis: (a) 2D model (FE thickness shown up), (b) 3D model.

Table 1 : Material parameters assumed in analyses.

	Unit weight [kN/m^3]	Modulus of elasticity E_x [GPa]	E_y [GPa]	E_z [GPa]
Masonry arch barrel	20	12	30	30
Other masonry elements	20	12	12	12
Backfill	18.5	0.3	0.3	0.3

Results of calculations by means of the 3D model utilizing the parameters given in Table 1 are shown and compared with measured quantities in Fig.10. Finally, the convergence reached between the calculated and the measured vertical displacements of selected points of span no. 2-3 (see Fig.3) were considered as satisfactory.

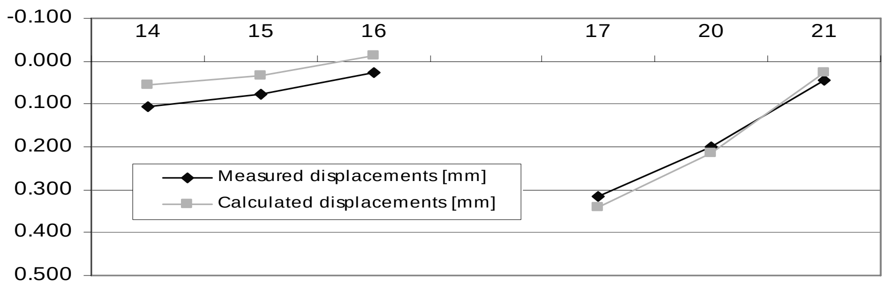
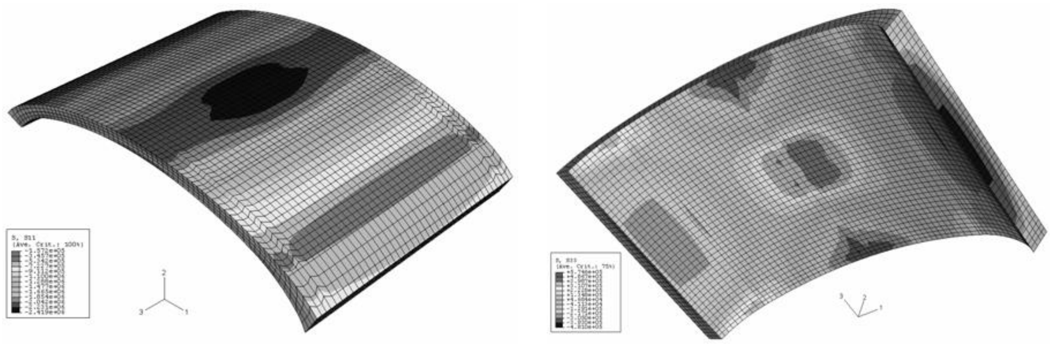


Figure 10 : Calculated and measured vertical displacements of selected points of span no. 2-3.

In the next step the simplified 2D model was applied for evaluation of the level of stresses and displacements in the whole structure as well as for selection of the span with critical cross-section taking into account code loads. Dead loads caused the highest peripheral tensile stresses in span no. 2-3 and the highest compressive stresses in span no. 4-5 while live loads provided similar level of peripheral stresses in all spans.

Taking into account the results of calculations for the final analysis related to assessment of the bridge load carrying capacity the span no. 2-3 was selected. In these calculations the 3D model was again utilized. This model, unlike 2D model, enabled variable application of the live load over bridge cross-section and therefore precise representation of the code loading could be defined.

Selected results of calculations presenting stresses in the main (peripheral and transverse) directions in the arch barrel of span no. 2-3 caused by code load corresponding to C class of the Polish standard are shown in Fig.10. Presented stresses are triggered by the most unfavourable location of loads including code vehicle placed centrally in the middle of the span. The maximum compressive stresses equal to 2.4 MPa appeared in the peripheral direction on the top surface of the arch (Fig.11a) and did not exceed assumed material strength of 5 MPa (calculated according to Eurocode 6). The maximum tensile stresses equal 0.58 MPa were detected in the transverse direction on the bottom surface of the arch (Fig.11b) and almost reached the corresponding material strength (compare Van der Pluijm 1997). This susceptibility of the arch to its bending in the transverse direction was also confirmed by presence of the longitudinal cracking (shown in Fig.4a) in the middle of a few arches.



(a) (b)

Figure 11 : Analysis results for the arch barrel of span no. 2-3 with the code load applied: (a) peripheral stresses σ_{11}, (b) transverse stresses σ_{33}.

Influence of the defects on the structure load capacity has been assessed by means of the expert tool MyBriDE (Masonry Bridge Damage Evaluator) based on hybrid network technology (Kamiński 2008). In this way the current load carrying capacity was established at level of C class of Polish code PN-85/S-10030 which permits crossing the bridge by vehicles with total weight of 30 tonnes.

6 SUMMARY AND CONCLUSIONS

The Osobowicki Bridge is a representative example of an old masonry bridge structure suffering from typical problems in contemporary exploitation conditions. As it in many cases happens, there was no detailed documentation of the structure available and all information describing the bridge had to be reconstructed (Adamcewicz et al. 2009).

Technical condition of this over 110-year-old bridge is relatively good although some typical defects are present. One of the biggest problem of this kind of structures is an insufficient waterproofing related simply to its aging and causing further damage like degradation of the structural material or cracks and losses.

Next to degradation of the technical condition another problem for such an old structure is increase, within the last decades, in permanent and heavy traffic loading of the bridge which was designed and constructed according to past operation conditions.

An interesting finding coming from the performed analysis of the bridge is that the increased loads do not threaten the strength of the arch in the longitudinal direction but become important taking into account the transverse response of the arch. This effect is related to a relatively low load capacity of the arch barrels of wide bridges (working like a plate) against their capacity in the longitudinal direction (working as an arch where compressive axial force plays a relevant role). A confirmation of this effect are longitudinal cracks detected during the survey of the structure (shown in Fig.4a) in the middle of the arches' width. Another aspect of the structural behaviour is a low influence of spandrel walls on the stiffness of the arch barrel. This effect is visible in Fig.9 where displacements of the edge part of the arch (in points no. 14 and 17) are the largest and only slightly reduced by the spandrel walls.

A general conclusion from FE analysis of the bridge is high effectiveness of the applied numerical models in evaluation of the load carrying capacity. Also a relatively low importance of exact evaluation of geometrical and material parameters was found considering possibility of receiving satisfactorily precise results.

Among the most urgent maintenance works the following tasks were indicated:

(1)refilling of the losses in road surface and pavement,

(2) removing of contaminations of supports and bottom parts of arch barrels,

(3) reconstruction of selected elements of external devices on the bridge including thermal covers of pipelines.

More serious repair works requiring to be performed within the next two years are:

(1) renovation of arch barrels including injection of cracks and fractures, refilling material losses and removing superficial sediments,

(2) elimination of the transverse displacements of spandrel walls with their renovation and more efficient connection to arch barrels,

(3) constructing of a concrete deck uniformly transversally distributing loads to arches,

(4) refilling of cracks and losses of bridge supports,

(5) reconstruction of the waterproofing layer and a new drainage system,

(6) replacement of the road and pavement surfaces,

(7) removing of the pipeline system attached to the structure excessively loading the spandrel walls and marring aesthetics of this beautiful historical bridge.

Taking into account monumental character of the structure consultancy with the city architect regarding all the mentioned works is necessary.

7 ACKNOWLEDGEMENTS

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