CONSTRUCTION AND MONITORING OF THE OPARNO VALLEY BRIDGE

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SUMMARY

The bridge crossing the Oparno Valley is a classical concrete arch structure composed of the two independent bridges carrying a four lane motorway D8 from Prague to Dresden. The bridge is located close to the city Litoměřice in a hilly landscape which is a protected natural area. It made the construction complex because of a limited access to the site. The paper describes the construction process and monitoring of strains and deflections during construction. The second part of the paper deals with numerical analysis and comparison of the measured and calculated values. Also different prediction models for creep and shrinkage were compared.

Keywords: *Arch bridge, cantilever construction, monitoring, surveying, numerical analysis, prediction models, creep, shrinkage.*

1. INTRODUCTION

The Oparno Valley Bridge carries a 4 lane motorway D8 connecting Prague and Dresden. The bridge is composed of the two independent structures, each for 2 lanes. The span of the arches is 135 m, which is at the moment the second longest span on the concrete arch in the Czech Republic. The bridge is located in the protected natural area, which made the construction even more complex. The approach to the valley was not allowed and the bridge had to be erected from both abutments and the parts were connected in the middle of the arches above the valley.

The design of the bridge is described in detail in the paper written by Kalný et al. [1]. This paper deals with the construction of the bridge and with the monitoring of strains and deflections of the superstructure during the construction process.

2. CONSTRUCTION OF THE BRIDGE

2.1. General comments

Construction of the bridge was significantly influenced by local conditions. The site was situated in the protected natural area. The space available for construction was therefore very limited. Additional to that, the bottom part of the valley between footings of the arch was closed for construction activities. The construction site had to be divided into two separate locations – each at one end of the bridge and the building activities were organized symmetrically from both ends. This procedure was also necessary for a symmetrical loading of the arch during construction. On the other hand such procedure became rather expensive, since all the technological equipments (casting carriage for the arch, and movable scaffolding system used for casting of the bridge deck) had to be doubled.

2.2. Arch of the bridge

The arch of the opposite U shaped cross-section allowed for finding a good balance between the weight and load carrying capacity. The cantilever casting method (Fig. 1) of the arch required to develop a casting carriage for individual segments of the arch. The length of the segment up to 6 m was found as optimal, considering the weight of concrete and the length of the reinforcing bars. One half of the arch was composed of 14 segments.

A new casting carriage was developed by PERI and STRUKTURAS for this bridge. Its scheme is plotted in Fig. 2. Longitudinal steel beams beside an arch form the main carrying element. They were supported by 2 cross-beams which are anchored at the top surface of the previous segment. The formwork was suspended on the main longitudinal beams. In the front part of the carriage, there was a free platform which was used for the assembly of the steel reinforcement and for supporting of the bars overhanging in front of the actually cast segment. Under the carriage, there were two transversal bridges, which were used for curing of concrete after the movement of the carriage into a new casting position.

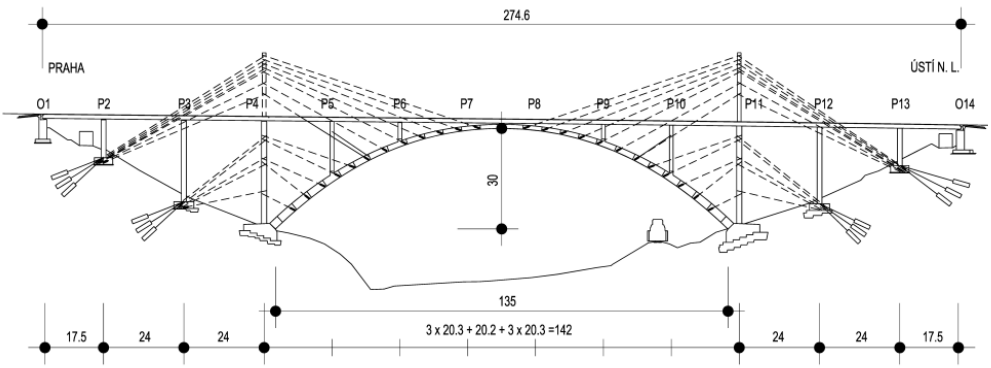

Fig. 1. Scheme of the cantilever construction of the arch.

The casting carriage was equipped by hydraulic systems, which allowed for an automatic movement of the carriage into a new position without the necessity of additional cranes or other devices.

The first two segments of the arch could not be cast using the carriage, since there was no space for its anchorage on the foundation. Therefore a classical formwork was used for the two segments and then the casting carriage was assembled and prepared for casting.

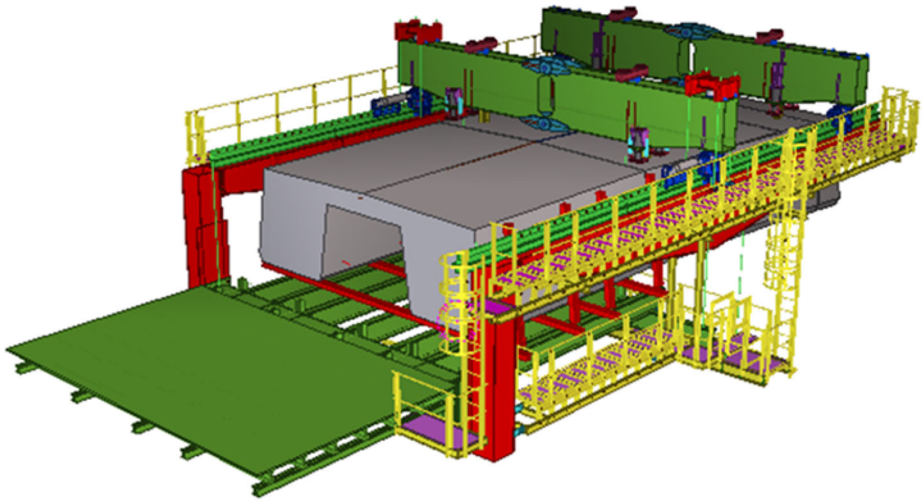

Fig. 2. *Scheme of the casting carriage.*

The casting procedure of the arch was not easy. The top surface of individual segments was in a slope and therefore a concrete mix had be either stiff, so that the surface could be kept without a formwork, or the top formwork had to be used.

The selected concrete mix and the casting procedure were tested on several trial specimens. The casting procedure was verified and the compaction technology was developed, so that a minimum of bubbles would be on the concrete surface. Finally it was decided to use the top formwork approximately on a bottom half of the arch, while the top half was cast without it.

During the casting of individual segments of the arch, the arch was subsequently suspended on temporary stays. When the cantilever became longer, a temporary concrete pylon on the pier above the footing of the arch was erected and used for installation of additional stays maintaining the stability of the arch. Before casting the closing joint in the top of the arch, the stays were used for final adjustment of the position of the individual cantilevers.

2.3. Bridge deck

Bridge deck is a continuous girder with spans in range of 17.5 to 24 m. The cross-section has a double T shape. After consideration of several alternatives it was decided to use an overhang movable scaffolding system (MSS) with two main top beams above the concrete deck. The main longitudinal beams are supported on special steel supports over the piers. At the ends of the main beams, there are long cantilevers which are used for the moving of the MSS into a new position. Above the longitudinal beams, the transversal frames carried the weight of the formwork and fresh concrete using the stays coming through the concrete deck.

2.4. Sequence of construction

After completion of foundations and piers outside the bridge arch, the bridge deck construction started from the both abutments. Contemporarily the arch was cast from the footings. The three spans, which were supported by piers outside the arch formed a tie and allowed for erection of the temporary pylon.

After closing of the arch, a part of the stays and the temporary pylon were removed and the MSS was used for casting of the central part of the bridge deck above the arch (Fig. 3). The casting procedure had to be symmetrical, so that the excessive stresses in the arch were avoided. When the first bridge was completed the equipment (casting carriage for the arch and MSS) were moved and the second bridge was be erected.

Fig. 3. *Construction of the bridge.*

2.5. Concrete technology

A special care was paid to the concrete technology. Casting of important parts was verified on concrete models. Casting in hot as well as in cool conditions was tested and appropriate conclusions were drawn, so that the superior quality of the bridge was achieved. The results of preliminary tests proved that the used concrete and the accepted precautions would work in accordance with expectations. An experimental testing as well as a numerical modelling helped the design and execution teams to guarantee the highest standard of the bridge.

The arch is a rather massive structure, where concrete of the strength class C45/55 was used. In order to eliminate high temperatures due to development of the heat of hydration, a special cooling system was developed. A system of tubes was embedded in the arch segments. The cold water from an external tank was delivered to the tubes. The control unit measured the temperature if in and out flowing water and adjusted the temperature and amount of the in flowing water. The temperatures were measured and the maximum values were limited to about 60°C. The cooling system was initially adjusted according to results of numerical analysis and according to the results of experiments; later the experience from casting of earlier segments was used.

3. MONITORING OF THE BRIDGE

With respect to challenging construction method the design and structural analysis of this arch bridge was quite extensive and complex. Diagnostic monitoring for measurement of strains and thermal fields in particular sections and surveying methods for setting and checking of executed structural shapes were proposed for continuous observation of the structures in progressing stages of construction process. For the left bridge, which was built the first, a detailed strain monitoring of the arch was provided in 13 sections in total – at the footings, at the crown, under all the piers of the deck and between them. For the right bridge a supervisory monitoring was provided only in a reduced extent. For the strain measurement 80 embedded vibrating wire strain gauges with thermal sensors were used. In each measured section there were 4 gauges deployed. In the reference bridge section there was also a set of thermal sensors located both across the arch and at the relevant deck section. The data were read as requested in the given schedule by the logger and transmitted daily through the integrated GSM module to the Client offices.

Fig. 4. *Vibrating wire strain gauge attached to the reinforcement of the arch.*

All measured strains and temperatures together with surveying data for the outlines of structural segments were used as one of basic sources for consecutive modifications of geometry during the construction process and for continuous evaluation of stresses in particular sections. The installed measurement set was also used for monitoring of strains during the bridge load test. Considering long-time measurements, the strain data were recorded in a limited extent. The surveying was interrupted, when the construction was stopped, however, when the bridge was finalized the observations were renewed.

4. EVALUATION OF MEASURED VALUES

The measured results were used in order to satisfy two objectives. First, the assumptions of the structural analysis should have been verified. The measured strains and deflections were compared with the estimated strains and deflections under the assumptions of the structural analysis. Second, the different models for creep and shrinkage were used in the structural analysis. The differences in numerical results were found. The comparison of numerical and measured values allowed for evaluation of the efficiency of individual prediction models. In order to make the comparison of numerical and measured values easier, a new computer program was developed by Kolínský [2]. A many times repeated analysis was required. It would be also possible to use commercial computer programs, but the developed program was made with a focus on the specific structure of the investigated arch bridge, which resulted in shorter computing time and easier comparison of results. The strains and deflections were calculated at the position of the embedded vibrating wire strain gauges, etc. The developed computer program PRAN (Parametric Rheological Analysis) is based on the analysis of a frame structure. The sections include the concrete part (taking rheological properties – creep and shrinkage – into account) and steel reinforcement, which is rather significant in the arch in particular. The rheological properties are modelled in the program using the rate type creep law, in the incremental form. The numerical models according to different recommendations and codes are transformed into the Dirichlet series (i.e. into the form of a chain Kelvin rheological model) which allows for a step by step analysis. The construction time and service life time is divided into time intervals, the strains, deflections and internal forces are analysed in individual time intervals. The results in the next time interval are calculated from those in the previous interval by adding appropriate increments. Such type of analysis is not new, it was developed by Bažant [3], but it allows for a fast and accurate analysis of large rheological systems, like the arch bridge under investigation.

The prediction models which were used for comparison were selected according to actual offer among the most advanced codes and recommendations. The model according to the Eurocode 2 (EN 1992-2) for bridge design was considered as a basic model, since it is widely used in current design practice. The most advanced model was developed in the fib Model Code 2010, which can be considered as an European model. The models developed by prof. Bažant and his co-workers were accepted officially by RILEM and are considered as the most advanced models developed so far. The older one B3 and the latest B4 take into account the precise composition of concrete and especially the model B4 also able to take into account the effect of admixtures used in contemporary concrete compositions Wendner [4, 5].The evaluation of measured and numerical values appeared to be a rather sophisticated task. It was necessary to obtain a lot of data from the records from construction. The construction sequence was possible to trace from the daily site records. However, e.g. the forces in temporary stays were very difficult to receive since they were modified several times and not all forces were perfectly measured. Some inputs like that had to be estimated indirectly from the known values and from the other data available on the site.

A numerical analysis followed a construction sequence as it was recorded in site documents. The results were focused on the arch construction which was the major problem of geometry and also strain and stress development. The comparison of measured and calculated values comprises the vertical and horizontal displacements of individual points on the arch and strains in the positions of the vibrating wire strain gauges.

4.1. Vertical displacements of the arch

Vertical displacements of the arch were measured during the construction very carefully. They were important for adjustment of the camber of the next segment and for the achievement of the correct shape of the entire arch. The geometry of each segment was measured several times: after casting, after movement of the casting traveller and after casting of the next segment. Due to the variable loading and due to adjustment of forces in temporary stays, the deflection of each segment is fluctuating. The similar figure was observed in the numerical results. Fig. 5 shows the measured and calculated values of vertical displacements of the segments 9A and 9B (left and right hand side of the arch). Total 14 segments were cast on each half of the arch, segment no 1 is at the footing, segment no.14 is at the top of the arch; the closing joint is located between segments 14A and 14B.

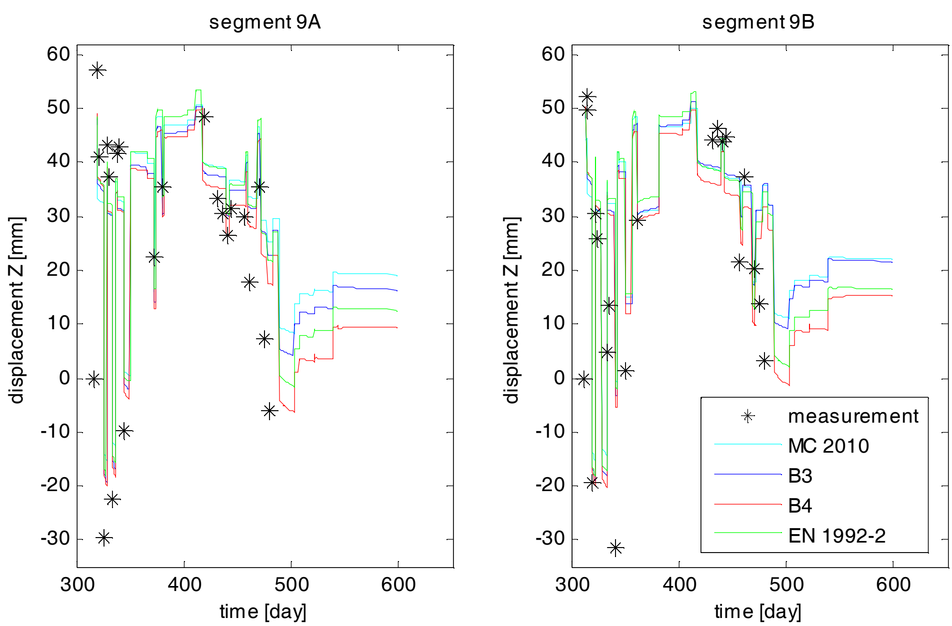

Fig. 5. *Comparison of measured and calculated vertical displacements of the segment 9A and 9B.*

A fluctuating deflection of the segments 9A and 9B can be clearly seen. Also a good agreement of measured and calculated values was observed. The numerical results during construction are very similar, for individual prediction models. The deflections are rather short-term and their values are determined preferably by the Young modulus of concrete. After longer time, the deviations between results according to individual models may be seen. However, at this stage the differences are rather small and remain hidden within the scatter of the measured deflections.

4.2. Strains in the arch

In each segment of the left bridge (erected first), there were 4 vibrating wire strain gauges embedded close to the top and bottom corners of the arch cross-section. The corresponding strains were calculated using the computer code PRAN. The strains were compared and they are illustrated in Fig. 6 which shows a typical variation of strains (segment No. 5B and segment No. 7B) during construction of the bridge. It may be observed, that the numerical analysis is able to fit the construction stages quite well and a reasonable agreement with measured strains has been achieved. In Fig. 7 a similar strain variation is plotted in logarithmic time scale for the entire service life of the bridge. Additional measured strains after completion of the bridge are included. It can be seen that the strains during construction are fit well using all prediction models. However, in the longer time interval, the model B4 provides better fit of the measured strains than the other prediction models.

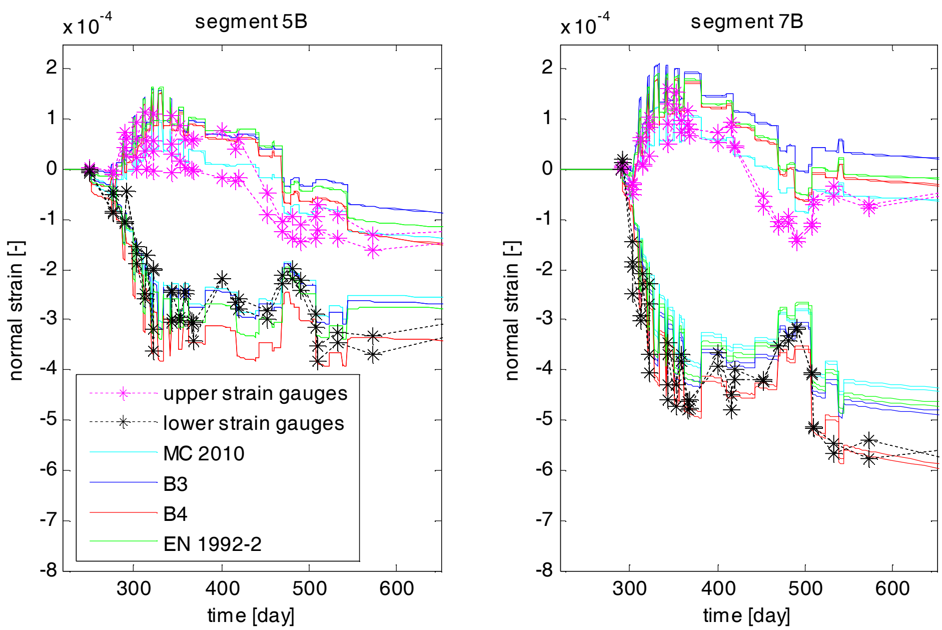

Fig. 6. *Typical variation of strains in segments 5B and 7B during construction.*

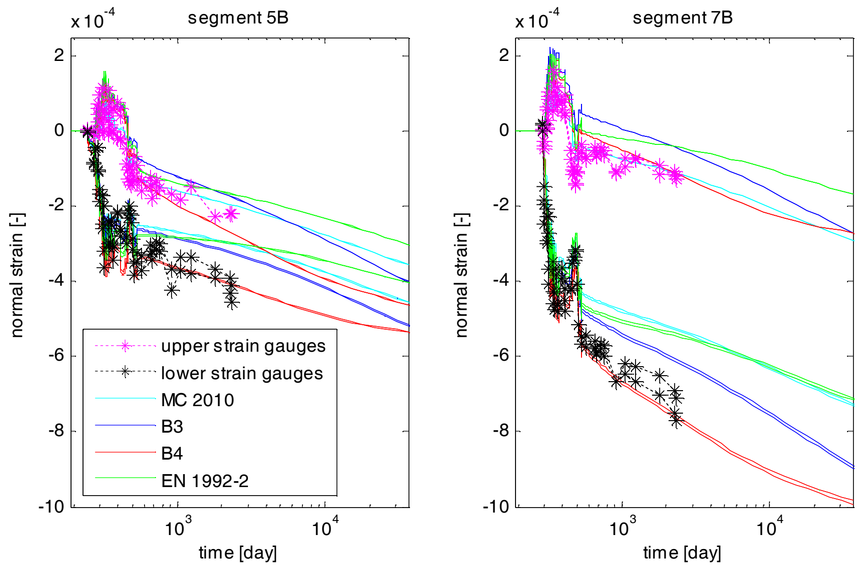

Fig. 7. *Typical variation of strains in segments 5B and 7B.*

5. CONCLUSIONS

The complex erection procedure of the bridge resulted in a structure which completely satisfies the requirements of the design.

The measured deflections and strains were modelled using specialized software. A reasonable agreement between measured and calculated values was obtained. Four different models for prediction of creep and shrinkage strains were used. The deflections and strains during construction are well fit by all used models. During the service life, the results using the B4 model, seem to show better fit of measured strains that results obtained by other models. However, the results may differ in different sections of the arch. In some sections, model B3 provided even better agreements of calculated and measured strains than B4. The analysis was carried out after completion o the bridge, when all the data (material data, time schedule, forces in stays, etc.) were available. If such analysis was carried out before the construction started, slightly different results could be obtained. The bridge was designed as a robust structure. It is sensitive to uncertain input parameters, like creep and shrinkage of concrete in this case, in a very limited extent.

Finally it is necessary to state, that the robust design of the bridge is correct and it may be expected that no large deformation or extensive redistribution of internal forces would occur during the service life of the bridge.

ACKNOWLEDGEMENT

The evaluation of the measured deflections and strains was carried out under the support of the Technological agency (Research centre CESTI, project no. TE01020168). The support of PONTEX, Ltd. (designer of the bridge), which provided the experimental data, is gratefully acknowledged.

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