LONG-TERM OBSERVATION OF BRIDGE STRUCTURES

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Summary: *For highway in the city of Žilina, the same twin parallel precast box girders were used with main spans of 60.5 m for a total length of 1042 m, where the same cross section was castoff throughout the bridge. The 60-ton segments were precast in an existing yard and delivered by trailers. They were lifted into place by a self-launching gantry placed on the previously completed portion of the deck. In order to assure the safety under construction of bridges, strain and temperature observation at the site are conducting since August 2016. In this paper, measured data are analysed. As a result, straining variation produced by temperature has revealed linear correlation, essentially simplifying design procedure. The data recorded for the study include detailed obvious measurement of the bridge structures during construction, along with measured strains and temperatures. Data significant for long term behaviour of structures will still being collected.*

Keywords: *Bridge, testing, strain, thermal effects*

1. INTRODUCTION

The paper deals with evaluation of experimental data collected during the continuous beam bridge construction and with subsequent analysis of the construction process and long-term behaviour with regard to environmental impact on precast concrete superstructures [1]. It is built using cantilever casting technology with self-launching gantry. The data recorded for this study include detailed geodetic measurement of the bridge structure during construction, along with measured strains and temperatures. Most of the data are measured during the bridge construction in 2016 and 2017. Data significant for long term behaviour of structure will still being collected. Verification of different concrete material models and their suitability for design of continuous segmental bridges built by free cantilevering will be a main result of the investigation. On the basis of a detailed comparison of numerical results and measured strains and temperatures, it is

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possible to quantify the impact of the individual input parameters on the resulting structural behaviour. The examined structure is made of prestressed concrete and consists of compact solid section. Basing on the measure value, the long-term deflection correction coefficient of long-term deflection can be introduced to predict construction deflection, more close to the practical situation. The long-term deflection calculation method based on the coefficient of correction can be also used in the prediction analyses.

2. STRUCTURAL HIGHWAY BRIDGE ARRANGEMENT

The new bridge composed of two separate concrete beam structures is located near the city of Žilina, approximately aligned in the south and north direction of European highway corridor. The superstructure of concrete class C 40/50 has been built with a length up to 1042 m between expansion joints. After consideration of the technical and architectural aspects in the urban area, it was decided that each bridge structural system would consist of a total of eighteen continuous spans with lengths 46.10 +15 x 60.50 + 49.80 + 32.80 m. This distance arrangement avoided spans of significantly different lengths. The abutment spans of bridges are only 54 to 76% of the central span length. Thus, these shorter end spans could minimize the length of the bridge adjacent to the abutment, which had to be built by employing falsework. The roadway has a variable width from 11.75 to13.25 m between barriers and the total deck width varying from 14.25 to 15.75 meters.

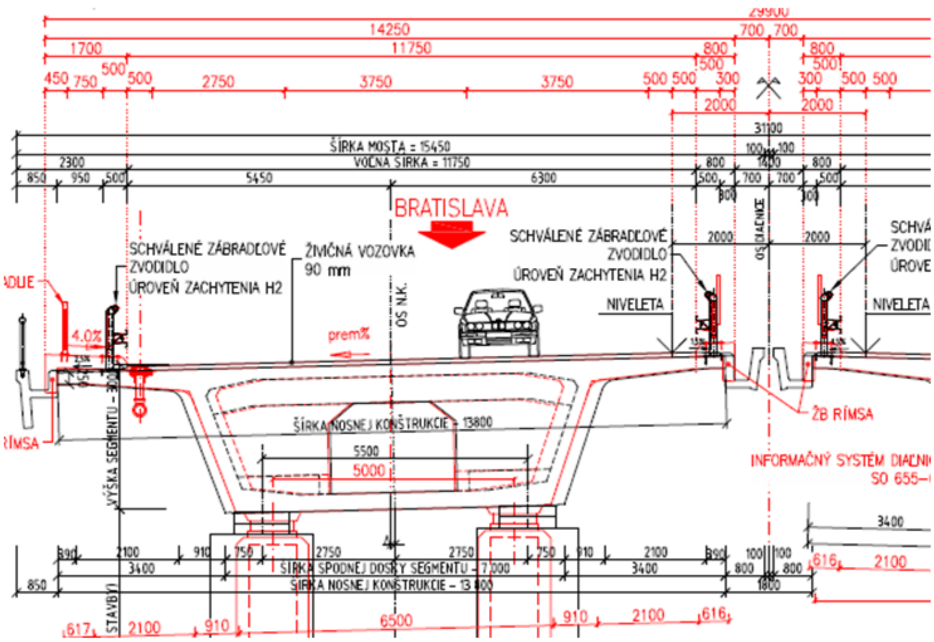


Figure 1. Highway bridge cross - section

Figure 1 shows the typical dimensions of a characteristic bridge cross-section consisting of single-cell box girders of the constant-depth 3.0 meters providing the most efficient

section for casting. Thus, the internal span-to-depth ratios are between 11 and 20. The inclined webs improved aesthetics, although introduced slight added difficulties in formwork. The web thickness 250 mm was determined by shear considerations, as tendon ducts internal to the concrete were present. The local haunches are used at the intersection of the bottom slab and the webs to provide sufficient space for accommodating the required number of tendon ducts. The distance 7.5 m between the webs at their intersection with the top slab was determined by achieving a reasonable balance between the moments and providing the necessary flexural capacity. The top slab thickness resulted from limit deflection criterium under live loading. Since segmental box girder has transversely post-tensioned top slabs, the minimum thickness should be 250 mm, with possibly thicker values at the tendon anchorages. The bottom slab thickness could be less, down to 200 mm, as there is no transverse post-tensioning embedded.
Two pairs of reinforced concrete column piers of square-shape section provide vertical supports for spans at intermediate points. In addition to transferring superstructure vertical loads to the foundations, the piers should resist higher lateral actions caused by potential earthquake events in this low seismic area and consequently ductility aspects had to be provided by their design. The seat-type abutments were constructed separately from the bridge superstructure as the reinforced earth-retaining structure using multiple-layer strips from nondegradable fabrics to reinforce the fill material in the lateral direction. The face panels as slabs anchored by the strips are subjected to lateral soil pressure. The bridge superstructure seats on the abutment stems through pot bearings comprising plain elastomeric disks confined in shallow steel rings. Teflon sliding surfaces of expansion bearings can accommodate translational movement. Keeper plates are used to retain the superstructure moving in presumed direction. The fixed bearings allowing only rotations but restricting movements are located at the top of the central piers. The placement of expansion joints within these long viaducts was necessary to accommodate the change in length of structure 560 mm due to creep, shrinkage, and thermal changes. The expansion joints are located at the centreline of the end abutments.

3. FABRICATION AND ERECTION

Precast segmental construction as the fast building system was applied due to restricted time for the erection. Moreover, this balanced cantilever segmental construction allowed building the concrete box-girder bridges without the need for falsework. This method had additional advantages especially in this municipal zone, where temporary shoring would disrupt traffic and services below, where falsework would not only be expensive but also a hazard. The major part of the work could be performed in the before now existing precasting yard, protected against inclement weather (Fig.2). The casting of the superstructure segments could start already at the beginning of the project and at the same time as the construction of the substructure, since the speed of erection was much faster than production output of the casting yard. Also, the segments were produced in an assembly-line factory environment, providing consistent rates of production and allowing superior quality control. The time-dependent deformations of the matured concrete are sure to be less important, as the concrete have reached a higher age by the time the segments were placed in the structure.

Figure 2. Segments casting in in adjustable molds

The successive segments were cast against the adjoining segment in the correct relative orientation with each other starting from the first segment away from the pier. The levels of accuracy in the segments match-cast against each other were sufficiently high in order to assure acceptable tolerances at the tip of the cantilevers. Geometry of the bridge in mutual space curvature is quite complex. To obtain a bridge with a vertical curve, the match-cast segment had first to be translated and given a rotation in the vertical plane. To obtain a horizontal bend, the conjugate unit was given a rotation in the horizontal plane. All these adjustments of the conjugate unit had to be combined for obtaining the desired geometry of the bridge.
Usually, the industrialized execution of the structure provided higher quality of the finished product, even though requiring relatively important investments in molds, lifting gear, transportation, and erection equipment. But volume of work validated the precast segmental building procedure as economically viable.

Figure 3. Self-launching gantry with centre leg on pier transporting a segment

Construction has commenced from the permanent piers and proceeded in cantilevering out to both sides in such a way that each phase was tied to the previous ones by post-tensioning tendons, incorporated into the permanent structure, so that each phase serves as a construction base for the following one. The structure was hence self-supporting at all stages. Nominal out of - balance forces due to loads on the cantilever were resisted by propping against an overhead gantry. The cantilevers were mostly constructed in 2.2 m long segments. The size and weight of precast segments 60 tons were limited by the capacity of transportation and placing equipment. These precasted segments in the casting yard and molds were transported to the specific piers by land, than on the completed viaduct, in the same order, and hence no adjustments were necessary between segments during assembly. The joints were made of the very thin layer of epoxy resin, which did not alter the match-cast geometry. Posttensioning could proceed usually as early as

practicable since there was no need for joints to cure. Post-tensioning tendons were internal to the concrete section housed in pipes. A minimum number of tendons were required for the balanced cantilevering process, anchored on the face of the segments and internal blisters.
The single erection truss with portal legs was used as the lifting equipment. This self-launching gantry has a length slightly longer than the typical span (Fig.3). During erection of the cantilever, the centre leg rests on the pier while the rear leg reposes on the cantilever tip of the previously erected span, which must resist the corresponding reaction. Prior to launching, the back spans had to be made continuous. Then, the centre leg was moved to the forward cantilever tip, which had to resist the weight of the gantry plus the weight of the pier segment. This stage controls the design of the gantry, which must be made as light as possible, and of the cantilever.
The several actions had to be considered in checking verification. Firstly a possible out-of-balance one segment on the cantilever. Then the presence of a stressing platform, live loading on one side of 1.5 kN/m^2, wind loading acting during construction and the possibility of one cantilever having a 2.5% higher dead weight than the other [2],[3].

4. SUPERSTRUCTURE TESTING

The compressive strength as the common performance property was measured by breaking cylindrical concrete specimens 150. 300 mm size (Fig. 4a) in the laboratory compression – testing machine and calculated from failure load divided by the cross – sectional resisting area. The results from cast cylinders validated existing concrete strength as well as adequacy of curing and protection measures. The next test method measured the load-induced time-dependent creep strain on moulded concrete sample subjected to sustained longitudinal compressive load (Fig. 4b). Creeping testing machine consists of main unit using disk springs specifically selected to maintain constant load over the range of deformation probable during creep tests and measured using externally mounted mechanical gage points.


(a)

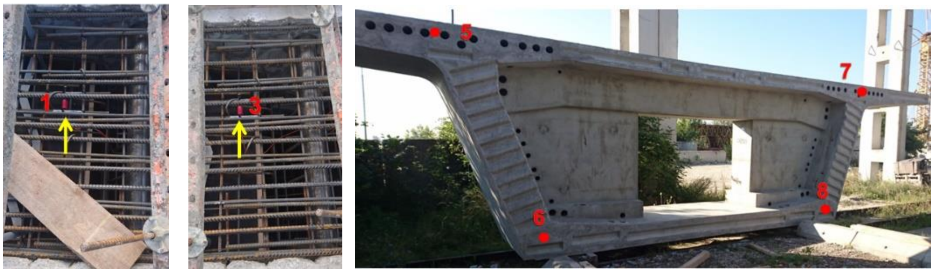

(b)


(c)

Figure 4. Concrete strength (a), creep (b) and strain (c) testing tools

Temperature gradients are caused by the top or bottom surface of the structure being warmer than the other. This distribution are nowadays assumed to be nonlinear with

magnitudes given in relevant standard [4]. Due to its high thermal mass, concrete structures are more adversely affected by the thermal gradient than steel structures. Effects of a linear portion of temperature gradient impose the unrestrained curvature at any point along the span. At that point, the final force distribution can be determined by evaluating the redundant support reactions. Nonlinear constituent of temperature gradients are more difficult to evaluate. The free thermally induced strain is proportional to the temperature distribution. In order for the section to remain plane under the effects of the applied temperature gradient, the self-compensating stress is induced. The final strain distribution on the section is the sum of the free thermally induced strain and the strain induced by the self-compensating stresses. Similar to the linear gradient, the total stress on a section is, therefore, the summation of the self-compensating stresses and the continuity stresses. Real more appropriate temperature distribution is the aim of our field measurements.
For testing, the seventh and sixteen mid-spans as well as two neighboring piers sections were implemented with sensors and tested. The vibrating wire strain gauges (Fig. 4c), suitable for long term readings were placed on the concrete deck and on the girder web [5]. Four strain gauges were placed at the intersection of the slabs and the webs of cross – sections and spaced along the length of each girder. Since there were two girders, a total of 32 strain gauges were placed on both bridges. The histories of stran development were recorded and structural response computed from the measured readings. At the same time, temperature measurements were made on the specified 32 different locations across the center of the span and pier sections. The data acquisition for each site investigation took about six hours. Figure 5 illustrates a cross-section view of the bridge and the detail distribution sensors. The bottom thermometer were attached to the outside of the girder at the mid height of the web. The bottom indoor sensor were located on the inside bottom flange of the girder. The top outdoor thermometer were located next to the concrete curb at the end of the deck. All sensors were protected from the direct contact with sunshine either by the bridge itself or by the shades made from duct tape and cups.



(a) (b)

Figure 5. Wire strain gauges locationsion in mid-span (a) and pier segments (b)

The first measurement was executed on August of the last year. The air was dry throughout the test. In addition to the temperature effect, traffic, winds, construction work on the bridge and other environmental conditions might produce changes of the measured parameters. However, since the bridge was not used and the weather was calm during the test period, it could be assumed that any changes of the parameters are mainly the result of the temperature changes. It could be presumed indeed that the temperature changes of the bridge were mainly responsible for the variation of the strains. This assumption seems reasonable since the bridge was not in service and there was no significant change of weather conditions on the test day. Thus, the measurement technique has separated variations caused by temperature changes and those produced by structural change or other environmental effects. Observations of the bridge data coupled with detail examination can led to the additional assumptions that appear simplistic but important factors in the design. Firstly it can be concluded that strain changes are linearly proportional to changes in temperature, as the straining variation were found linearly correlated with temperature readings from different parts of the bridge. Also it was observed that the mass of the bridge forced the strain change to delay behind the temperature and the bridge takes some time to warm up and cool. The geographical north-south orientation of the structure with respect to the sun originates that the temperature of the west end of the bridge will have a time-lag behind the temperature of the east end. Given these assumptions, a linear temperature distribution can be chosen in a system analyses.

5. CONCLUDING REMARKS

Global vertical deflections of segmental box-girder bridges due to the effects of dead load and posttensioning as well as the long-term effect of creep were predicted during the design process by the use of a computer analysis program. But the deflections are dependent, to a large extent, on the method of construction of the structure, the age of the segments when post-tensioned, and the age of the structure when other loads are applied. It can be expected, therefore, that the actual deflections of the structure would be different from that predicted during design due to changed assumptions. Real more correct deflection distribution is the next purpose of our field measurements. As a final point, the deflections will be recalculated, based on the actual construction sequence.

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ДУГОТРАЈНО ПРАЋЕЊЕ МОСТОВСКЕ КОНСТРУКЦИЈЕ

Rezime: *За аутопут у граду Жилина, коришћена су два идентична паралелна предфабрикована кутијаста носача, са главним распонима од 60.5 m и укупне дужине од 1042 m, при чему је исти попречни пресек постављен по целој дужини моста. Сегменти тежине 60 тона су изливени на постојећем градилишту и превожени камионима. Подизани су на своја места до претходно комплетираних делова моста самопокретним порталним крановима. Да би се осигурала безбедност испод конструкције моста, константно су праћене релативне деформације и температура на градилишту од 2016. У овом раду су анализизирани подаци добијени мерењем. Као резултат, добијена је линеарна корелација промене релативне деформације и температуре, чиме се суштински упршћава процес пројектовања. Подаци који су регистровани у сврху ове студије, укључују детаљна очигледна мерења контсрукције моста у току градње, заједно са мереним релативним деформацијама и температурама. Додатне информације, које су релевантне за дуготрајно понашање структуре, ће бити регистроване и убудуће.*

Кључне речи: *Мост, тестирање, релативне деформације, термички утицаји*