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VERIFICATION OF FLEXURAL BEHAVIOR AND SIMPLIFIED MODELING OF STEEL-CONCRETE COMPOSITE BRIDGE

Abstract

An experimental verification of actual flexural behavior of composite steel-concrete girder bridge is presented. The comparison of the experimentally obtained values with the values calculated using suitable computational model is also given in the paper. Introduction of changes in stiffness of concrete slab due to concrete cracking into the global analysis is discussed, too.

Keywords

Steel-concrete bridge, experimental measurement, result comparison, real behavior, simplified modeling.

1 INTRODUCTION

With the use of modern structural analysis computer programs, the most reliable design alternative, providing the most probable response of a bridge structure due to a range of designed loads, can be identified. Even though, many simplified technical approaches are routinely used in the application of theories to practice during design and analysis process of bridges. Moreover, there is usually lack of required time to verify all details. Therefore, a proof-load test is useful in certain circumstances, [1]. The main purpose is not to verify final design of the bridge but also to validate adopted assumptions of the designer. Actual reserves in load-carrying capacity of the new bridge structure can be determined after test evaluation, [2]. Thus, proof-load test supported by finite analysis model might represent the most powerful tool for verification of real behavior of bridges, [3]. The aim of presented research, whose partial results are introduced in this paper, was to verify the actual flexural behavior of a composite steel-concrete bridge.

2 ANALYZED BRIDGE STRUCTURE

The research dealt with a road bridge shown in Fig. 1 built across a highway. The analyzed superstructure was manufactured as a four-span continuous composite steel and concrete structure, [4]. Because of an angular crossover and arch curvatures of side road approaches, theoretical spans of left and right main girders are not equal. The left main girder has spans 17.483 + 31.249 + 28.812 + 24.279 m, while in the case of right girders the corresponding values are 24.489 + 31.156 + 28.848 + 17.684 m, Fig. 1. Moreover, the deck of the bridge follows the vertical arch curvature of the road on the bridge, as well.

The bridge superstructure consists of the reinforced concrete deck composed with the two plate girders of I-section axially 4.0 m spaced. The structural depth of the girders with the basic value 1300 mm in midspan regions is increasing within the 6.5 m long linear haunches on both sides up to 1800 mm above intermediate supports. Webs of the plate girders are 12 mm thick in the span areas and 16 mm thick above supports, respectively. To save material, variable area of both flanges

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proportioned to the longitudinal course of bending moments was used. The top flange acting with the concrete deck is of the constant 350 mm width with the varying thickness from 25 to 50 mm. The bottom flanges of 650 mm width have thickness from 30 mm in the span areas to 40 mm above the piers.

Fig. 1: Overall view on the bridge (top) and top-view on the scheme of superstructure (bottom)

Truss cross-frames consisting of horizontal chords and diagonals made of HEB sections ensure the lateral stability of the plate-girder bridge and help to distribute the vertical loads. As end cross-beams a welded I-section of 1000 mm height was designed. Low-alloy structural carbon steel of grade S355J2 has been used for steel bridge structural elements.

Reinforced concrete of quality C35/45 was used in the slab. The slab is 332 mm thick in the middle part with haunches towards the girders. In the outer parts, thickness of the slab decreases forms the value of 425 mm above the girders to the 207 mm at the ends of side cantilevers.

Shear stud connectors ø19/150 from steel grade S235J2 at the interface between the concrete slab and structural steel should ensure a full composite action.

3 EXPERIMENTAL INVESTIGATION

3.1 Measured values

During testing, the main girder's deflections in each span as well as the bearing settlements were monitored. In addition, the extra experimental investigation using 20 strain gauges was carried out. The strains in two selected cross-sections were observed in the flanges of main girders, in the concrete slab and in the bottom chord of bracings, respectively. The section almost in the middle of the longest (2nd) span was chosen in sagging moment area, just in the point, where the middle intermediate bracing is joined. For monitoring strains in hogging moment area, the same amount of gauges was installed in the bridge's cross-section in distance of 400 mm from theoretical support above the middle pier. The arrangement of strain gauges in cross-section and their denotation are shown in Fig. 2. Several of them are also visible in Fig. 3.

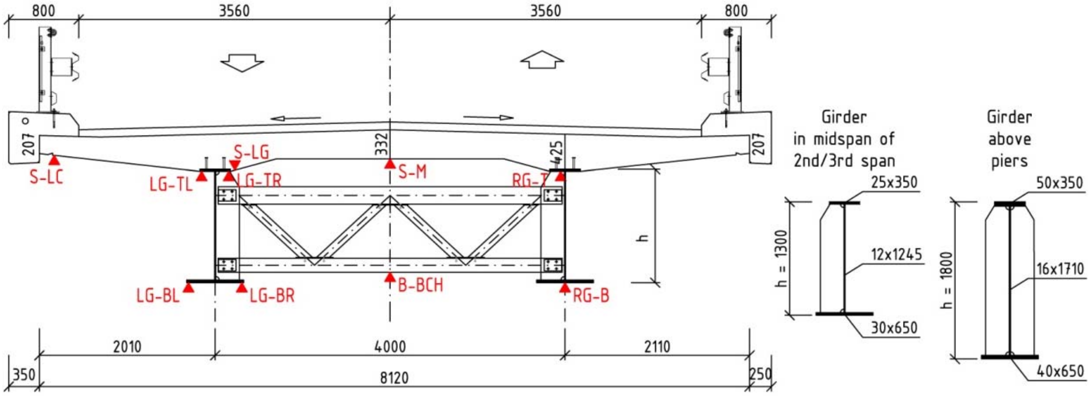

Fig. 2: Cross-section of the bridge and strain gauges

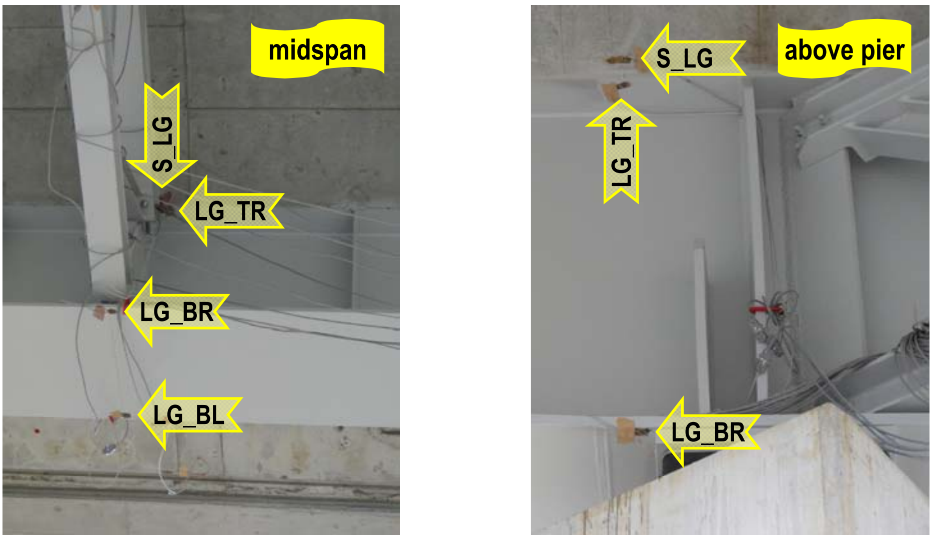

Fig. 3: Real position of several gauges in the "midspan" and "above-pier" cross-section

3.2 Testing load

For the purpose of the test, eight trucks Tatra 815 with the average gross-vehicle weight of 28.0 tons with deviation of ±2.0% were at disposal. Four load positions (load cases) represented by the group of these trucks were considered.

According to Fig. 4, load arrangements LC-1 to LC-3 consisted of the group of 5, 8 or 7 trucks, respectively, and were placed within a span in order to cause the maximum stressing and deflection of the loaded span. Actually, the applied test load represents load efficiency $\eta = 0.75 \sim 0.97$ in deflections and $\eta = 0.70 \sim 0.81$ in bending moments as compared to the values caused by traffic load given in the Eurocode 1.

The last load case LC-P consisted of two groups of four vehicles situated along the bridge axis in the adjacent spans to the middle pier. In the case of support moment above middle pier, the load efficiency of such arrangement is some $\eta = 0.74$.

A correct position of loads was determined on the basis of influence surfaces of the deck investigated on spatial finite element models described in the next chapter.

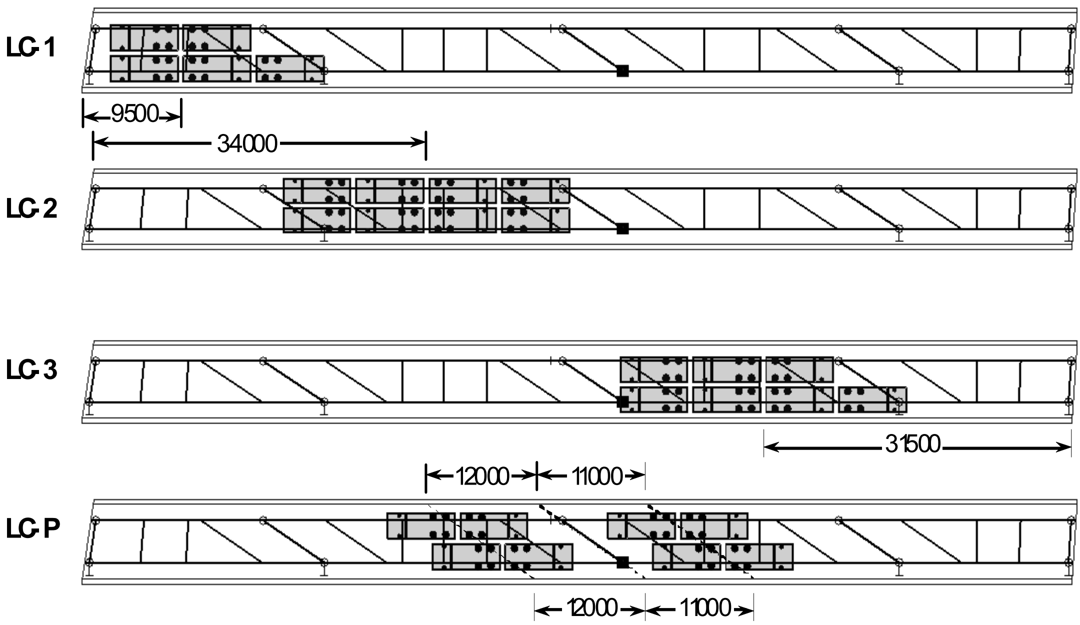

Fig. 4: Load cases - arrangement of lorries on the bridge

Fig. 5: Photography of load case LC-3 from left side

4 GLOBAL ANALYSIS

In the presented first stage of evaluation of observed data, common FEM-based software was used. A spatial numerical model combining plate and beam elements was chosen. Both the concrete slab and the steel girder were approximated considering variability of thicknesses and heights. Internal truss diaphragms and end cross-girders were considered as the beam elements respecting their characteristics including appropriate eccentricities.

As simplified modeling was the issue, no material nonlinearities were adopted into the analysis. Similarly to the simplified method given in [5], the effect of cracks in concrete was taken into account by neglecting the concrete in some area above the intermediate supports. Four concepts of modeling the concrete slab in the hogging regions were analyzed in the study. In the first one, the invariable flexural stiffness of the composite cross-section along the bridge length was thought

(EI _uncracked). The second model allowed for stiffness changes due to concrete cracking in the hogging regions using simplified approach according to [5] (EI_cracked). The last two models (EI_semi A and EI_semi B) came from an estimation of concrete stiffness somewhere between the borders represented by the "cracked" and the "uncracked" analysis.

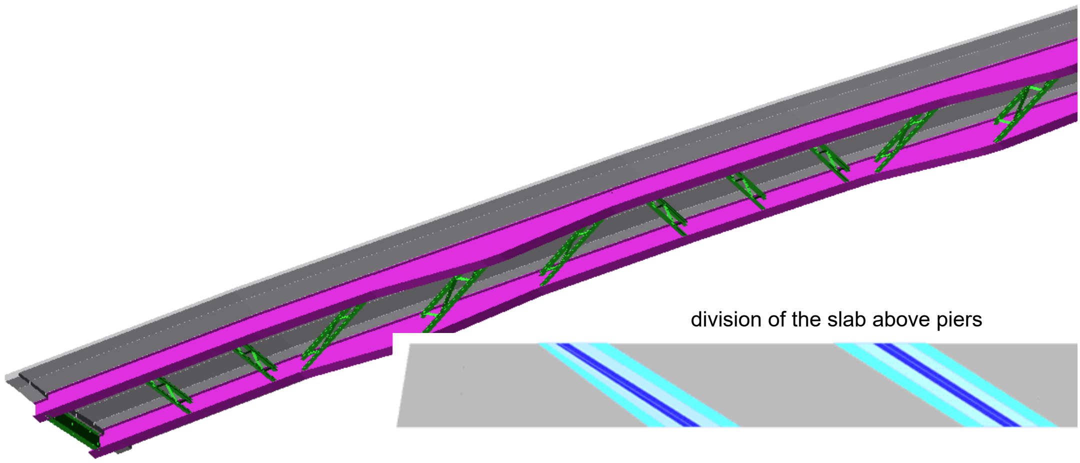

Fig. 6: Left half of FEM model of the superstructure
and highlighted division of the slab above piers

Thus, if cracks in concrete are taking into account, stiffness *EI* of composite sections should be reduced. In presented simplified models, linear analysis was applied. The stiffness reduction was made by modification of modulus of concrete material $E_{c,model}$ used in the transformation models. In the Fig. 7, the values of slab modulus $E_{c,model}$ introduced into the numerical model are illustrated as percentage of modulus of "uncracked" reinforced concrete slab E_{c+s}

$$E_{c+s} = E_{cm} + \rho \cdot E_s \tag{1}$$

where:

E_{cm} – is modulus of elasticity of concrete [N/mm^2],

E_s – is modulus of elasticity of reinforcement [N/mm^2],

ρ – represents reinforcement ratio [-].

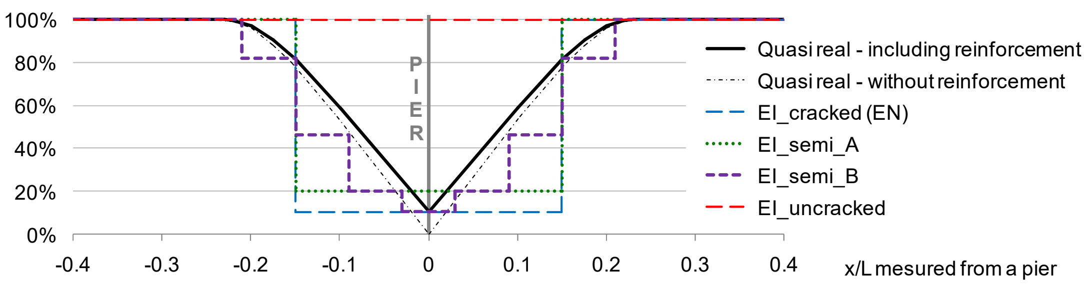

Fig. 7: Modification of modulus of slab $E_{c,model}$ in numerical models in percentage of E_{c+s}

It should be noticed that flexural behavior of the structure can be influenced by non-structural members of the bridge, as well. For instance, the cornices together with the steel handrails should be implemented into an improved transformation model, as they act as side beams of the concrete slab. The influence of cornices depends on cornices' anchoring system and on concreting system and phases, especially. Particularly in the winter time, some stiffness of the bitumen layers can be considered, as well. Since this article is dedicated to comparing experiment with simplified modeling approach, in our case, we did not consider above mentioned effects.

4 RESULT COMPARISON

Only small part of results is presented in the paper. Anyway, conclusions are based on the critical analysis of many other results, as well.

4.1 Deflections

Comparison of the numerically obtained deformation of the girders with those observed during measurement is shown in Fig. 8. The values valid for mid-span of the second and the third bridge span are confronted. Only three load cases are presented in the Fig. 8. Comparison of girders' deflections indicates that in the case of analyzed bridge, the "uncracked" analysis can provide results close to the measured values. The other models with less concrete stiffness above support produced differences on both sides, with dependence on load position and analyzed span.

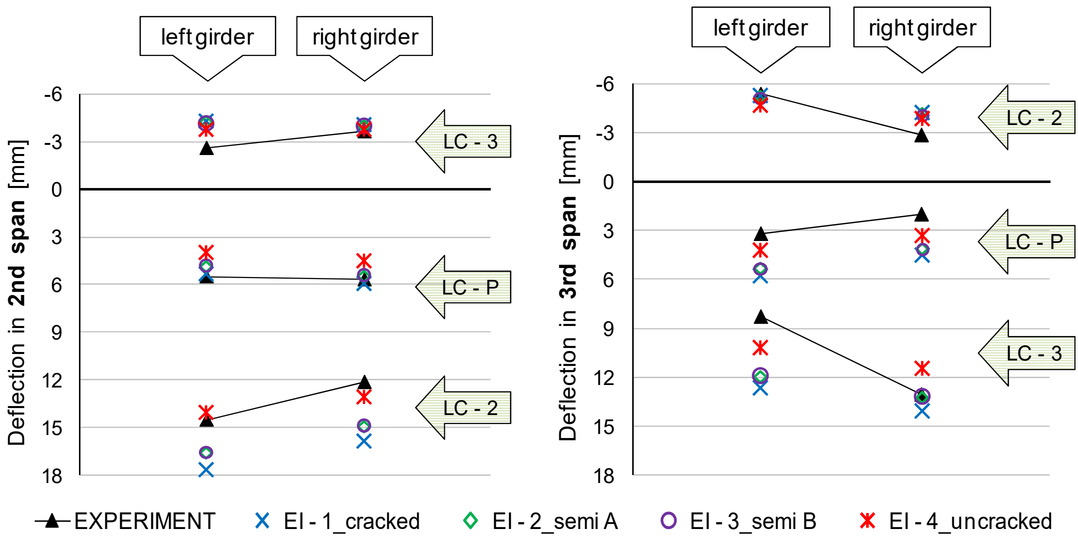

Fig. 8: Mid-span deflections in two adjacent middle spans produced by three load cases: results at the 2nd span are on the left-hand side and results at the third span are on the right-hand side

4.1 Strains/stresses

The strain measurements proved the elastic behavior of the composite steel and concrete bridge during testing.

In the next figures, the comparison of stresses in the steel girders expressed from the measured strains with the stresses obtained by means of the numerical calculations is presented. The stresses at left girder (LG) and right girder (RG) are shown through the height of corresponding girder section in the case of two load cases. The stresses calculated from measured strains in the case of track arrangement LC2 are put in Fig. 9, while Fig. 10 shows the values valid for the load case LC-P.

Presented values represent the stresses transformed from the data observed in corresponding gauges at the top or bottom flanges, respectively. In the case of two gauges glued on flanges of the left girder, the average value is given.

Unfortunately, a strain gauge glued to the bottom flange of right girder (the gauge RG-B according to Fig. 3) in the midspan cross-section got out of order during the test. Thus, only one measured point through the girder's height in the midspan can be found in that case in the Fig. 9 or. Fig. 10

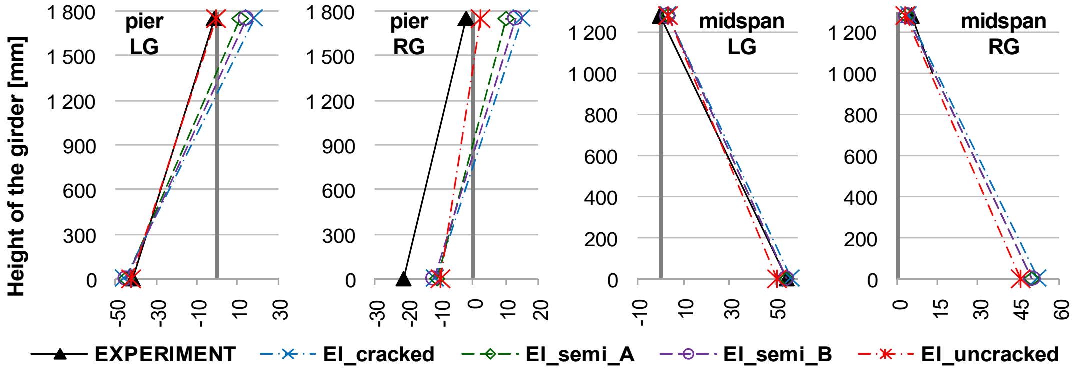

Fig. 9: Stresses through the girders height in the steel girders under load case LC-2

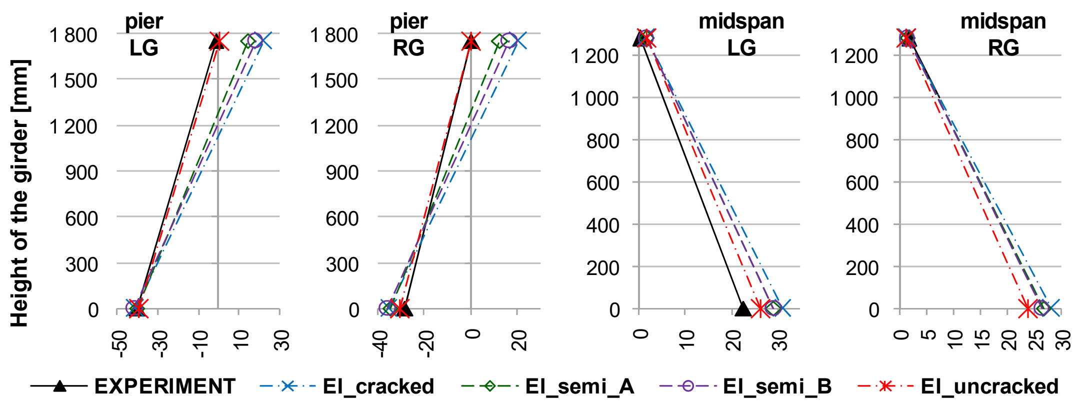

Fig. 10: Stresses through the girders height in the steel girders under load case LC-P

Similarly to deflection-based conclusion, it could be again stated that within the four analyzed models, the "uncracked" analysis gave the strain/stress results, which are closest to the observed ones. Analyses of the other three models with reduced stiffness above piers produced higher differences, especially in hogging moment regions.

The observed values of strains at the top of the girder in the intermediate support area indicates that either the concrete cracking has les influence on slab stiffness or the reinforced concrete can transfer more tensile forces than predictions coming from common assumptions of the codes.

4 CONCLUSIONS

The strain measurements proved the possibility of approximating the composite bridge by means of combined plate-beam model providing sufficiently accurate prediction of the superstructure behavior, especially the girders within span areas.

It could be concluded, that among the four analyzed models, the "uncracked" analysis with the constant stiffness of the reinforced bridge slab described the actual behavior of this composite bridge with the best accuracy, in general.

The stresses in the girders above intermediate supports are influenced by effects like concrete cracking, tension stiffening and reinforcement yielding. Allowing for these effects seems to be quite complicated without utilization of nonlinear analysis. A technique given in [6], when additional deformation loads supply effects of cracking in "uncracked" analysis, can be alternatively applied. Probably, next research will focus on this area.

However, in the phase of bridge design it is necessary to ensure the safe determination of the bridge response to action. In that case, the stiffness reduction in the hogging regions due to concrete cracking and tension-stiffening of concrete shell be given by the corresponding codes on conservative side to fulfill the requirement of the safe design of steel girder. Especially, in the case of two-girder bridge concept, when only one girder is loaded, the effect of the stiffness change in the hogging regions over internal supports is more striking, [3].

The presented experimental observations were done in the age of concrete of 50 days. Thus, majority of shrinkage strains had already been preceded. It would be useful to repeat the experimental measurement sometimes in the future, to compare obtained results. After decades, shrinkage effect will already be subsided and most of irreversible strains developed. Particularly, the effect of cracking and development of cracks produced by repeated loads would be also observed.

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