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NUMERICAL AND EXPERIMENTAL MONITORING OF THE SHEAR RESISTANCE OF SEGMENTAL STRUCTURES

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Research field: Design of Prestressed Concrete Bridges

ABSTRACT

The aim of this article is to describe a project which combines computer analysis with experimental measurement. The subject of this project is the Kishwaukee River Bridge, which has suffered from extensive cracking of it webs since its construction. The goal of the project is to assess the structure, determine the current status of the stresses in the structure and determine the ultimate capacity of the bridge.

KEY WORDS

- *shear crack*
- *crack width*
- *shear reinforcement*
- *shear stiffness, load test*

1. INTRODUCTION

The presented article is the result of a scientific and research collaboration between the departments of CME UIC at Chicago, Illinois, in the U.S.A and KBKaM SvF SUT Bratislava, which started in 1996. In 1980 a southbound bridge and two years later a northbound bridge were opened to traffic as two separate structures of the Kishwaukee River Bridge [5] in Rockford, Illinois. The bridges have post-tensioned precast segmental box-girder decks. The single-cell segments have one shear key in each web. In contrast to the northbound bridge the deck of the southbound bridge suffers from extensive cracking in its webs since its construction. The reason is more or less known. After completion of the bridge, it was disscovered that epoxy glue did not harden properly in most of the joints. A substantial part of the shear forces was concentrated at the shear keys.

After the failure of the SB1-N1 shear key, all the defective joints were repaired by using steel pins (Wang, Ming L. – Sapathi, Debashis – Lloyd, George M.: Monitoring & Damage Assessment of the Kishwaukee Bridge, March 1999). The smooth contact surfaces became indented (toothed) and substantially improved the transfer of shear stresses across the joints, but mainly for loads imposed after the retrofitting (barriers, the wearing surface and vehicular load). The steel pins have also enhanced the shear resistance of the joints. The disadvantage of the retrofit performed was that the structure had not been activated before.

The deck of the bridge has five spans with lengths of 51.8 m + 3 x 76.2 m + 51.8 m see Fig.1. The overall length of the decks is 334 m. The precast segmental decks were built by the balanced cantilever method. Each cantilever consisted of seventeen 2150 mm long segments and one 1067 mm long pier segment. The cast-in-place closures have a length of 984 mm. The cross-section of segments is

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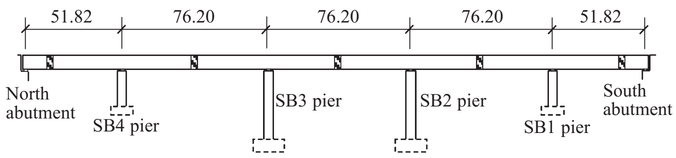

Fig. 1 *Longitudinal layout of the Kishwaukee River Bridge*

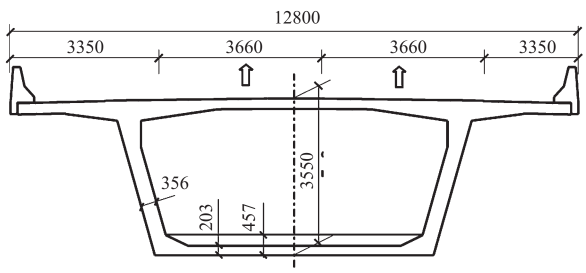

Fig. 2 *Cross- section of the bridge deck*

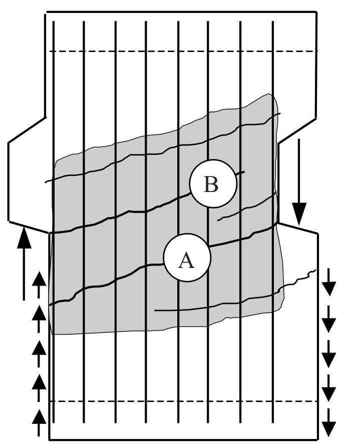

Fig. 3 *Typical crack pattern*

constant except for the first five segments from the pier, where the thickness of the bottom slab changes from 203 mm to 457 mm. The segments have match-cast epoxy joints with one shear key in each web. The compressive strength design of the concrete was 37.9 MPa. The decks are entirely prestressed by Dywidag high-strength threaded bars with a diameter of 32 mm located in the top and bottom slabs (no draped tendons are present).
The segments are reinforced by mild steel reinforcement grade 60. Each web contains eight stirrups of spacing a 254 mm. The stirrups were made from two 2ϕ22.2 mm bars in the first three segments next to the piers, two 2ϕ19.1 mm in the further three segments and, in the others from two 2ϕ15.9 mm. The longitudinal reinforcement consists of ϕ12.7 mm bars spaced at 254 mm at both surfaces.
The crack pattern in the webs is very different even within one segment (east and west webs). The angle of the cracks varies from 10° to 42°. The widest cracks are very flat, sloping at 15° and usually propagating from the bottom part of the female key (crack „A“ in Fig. 3) towards the next segment. In many segments, it can be observed that the crack propagates from the bottom part of the male key (crack „B“ in Fig. 3). These cracks are shorter and less wide than the former ones. The widest cracks are located next to the female key and have an average width of 0.75 mm. In the middle of some segments, they were found to be 0.65 mm. The most frequently observed crack width is 0.40 mm.

2. EXPERIMENTAL TEST

The main aim of the project was to determine the actual stresses in the structure, particularly the stresses in the shear reinforcement and secondly to verify the safety and durability of the bridge. Standard calculation techniques, could not be used because a very complex flow of internal forces had occurred in the webs during construction. Furthermore, the properties of the joints have significantly changed due to the retrofit performed. This has made the values of many of the variables needed for calculating stress unknown. In order to determine these values, a 50% scaled model of the segments was built.
The model consists of three experimental segments and simulates the behavior of the first three segments in the bridge (Fig. 4).
The model was made as a half-scale cut-out of the Kishwaukee bridge (Fig. 5) Instead of the originally inclined web, a vertical one

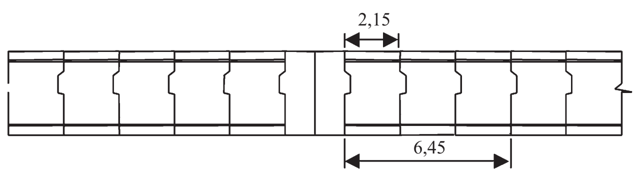

Fig. 4 *Modeled part of the bridge*

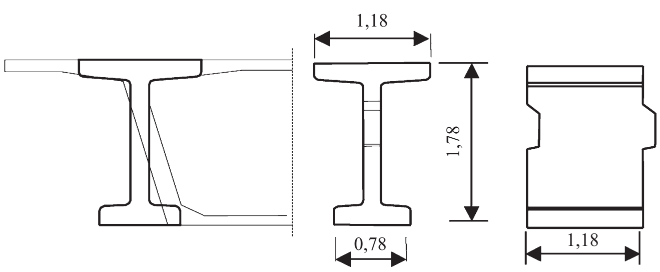

Fig. 5 *Model of segment web and its dimensions*

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was used. The width of the web of the model was designed from a dimension measured perpendicularly to the web surface. The lower flange is symmetrical. The centroid of the cross-section is in the same position as in the original segment.

The segments were cast from the B45 concrete grade, with a characteristic compressive cylinder strength of 35 MPa, which is in compliance with the strength of the concrete used for casting the actual segments.

The amount of shear reinforcement was established on the basis of the same shear reinforcement ratio of the actual and tested segments. The designed area was 0.00153, and it matched with 2f14 mm links and spacing at 200 mm. The male and female shear keys are reinforced by two f8 mm bars and their shape corresponds with the shape in the actual segment.

Steel 10 425 (V) was used as mild a reinforcement with a characteristic yield strength of 410 MPa, which is in compliance with the reinforcement grade 60 that was used in the bridge.

The arrangement of the experiment was adapted to simulate the method of bridge loading during its construction, because the first cracks had appeared during erection stage. The bridge was built by the so-called balanced cantilever method. Therefore, three experimental segments were fixed to the support acting as a cantilevered beam (Fig. 4). The prestressing consisted of eight three-strands tendons located in the top slab and four four-strand tendons located under the segments.

The designed prestressing ensured similar axial stresses as the prestressing units in the actual structure. The tendons located under the segments were anchored in a steel frame that leaned on the forehead of the first segment. Before prestressing, the joints between the modeled segments were treated by a thin layer of plaster in order to fill any voids and then painted with epoxy which made the surface very smooth. The treatment substantially decreased the friction coefficient and ensured similar properties of the joints with those in the actual structure (Figs. 6 and 7). The basis for the model's load arrangement was the loading of the bridge during construction. A step-by-step erection of the seventeen segments was supposed,

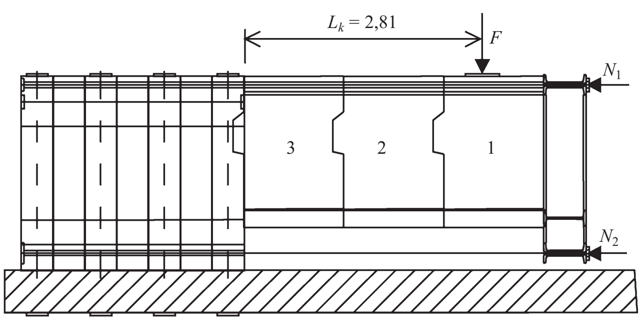

Fig. 6 *Arrangement of the Experiment*

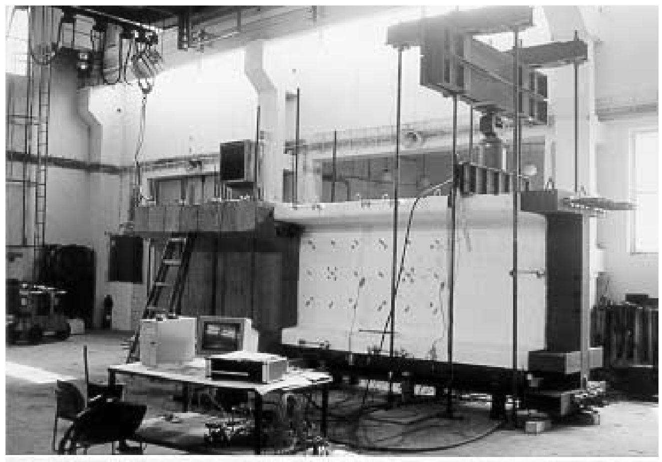

Fig. 7 *Arrangement of the Experiment*

with length of 2,15 m (Fig. 4) and effect of the weight of launching gantry (136 tons) was assumed. The loading of the model was adjusted with the intention of attaining similar stresses in segment #2 as were in the actual structure during construction. We were not able to model the same stresses for each segment, because of the constant number of prestressing tendons in each segment. In the real structure, the number of prestressing bars differs for each segment. The effect of the weight of successively erected segments was modeled by a 200 ton jack. The jack was located in the middle of the first segment. The magnitude of the force was controlled by a dynamometer. Because the period of construction was nearly half a year, the maximum service load was kept on the model for six months. During the experiment the 200 tons jack was exchanged for two prestressing jacks to increase the capacity of the loading equipment.

3. MEASUREMENTS

The model was loaded step by step in several stages. The following parameters were measured for each loading stage:

1. Strains in the top and bottom slab, strains in the web just behind the steel cross-beam for dispersal of force – strain gauges on fixed bases
2. Strains in the webs in longitudinal, transverse and inclined – diagonal directions, opening and closure of the joints – strain gauges
3. Deflection of the beam at the fix and the end of the support
4. Stresses in the reinforcement and stresses under the shear keys – tensometers.
5. Magnitude of applied force – dynamometer

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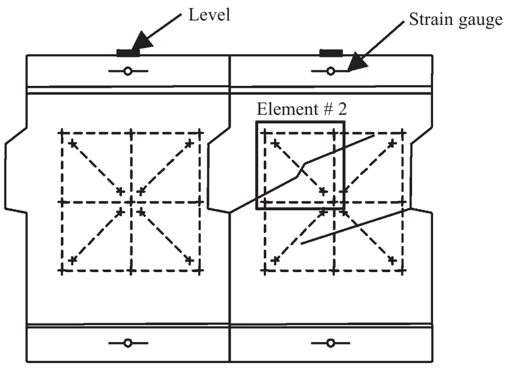

Fig. 8 *Measurement basis*

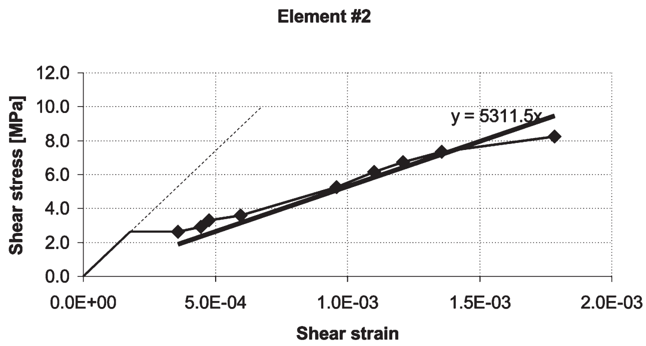

Fig. 10 *Shear stress - shear strain diagram*

6. Elongation of the strands during stressing
7. Rotation of segments #2 and #3 in a longitudinal direction and the transverse rotation of segment #1

The measurements began in May 2001. There were 75 measured steps. We did not use each measured record. The records were chosen for groups called Data Report (DR). There were three main data reports. DR1 represents the records measured from the beginning to the state , when the stresses in model segment #2 correspond to the stresses in segment #2 of the bridge, after finishing of the cantilever. The measured strains were generated by prestresing (N1,N2, see Fig. 6) and shear force F (see Fig.6). DR2 represents the records when the force F was incrementally increased to its maximum of F=1300kN. DR3 represents quick loading from zero to the maximum F=1300kN.
In Fig. 9, the force – deflection diagram originated by connecting DR2 to the end of DR1 and then DR3 to the end of DR2. The deflection were measured at the end of the cantilever. This system of evaluation was used for all the measured equipment's sets on the model.(see Fig. 8). The second type of results is in Fig.10, the stress-strain diagram for shear stresses and shear strains. From Fig.10 a decrease in shear stiffness is evident. The inclination of the dashed line is the theoretical shear modulus, Gir=14.8GPa. The shear stiffness reduction for element #2 (see Fig. 10) is 36%.

Nonlinear analysis

The results from the experiment were used to calibrate the model for nonlinear analysis. The analysis was performed by the FEM program ATENA 2D. The program uses plane elements, which means that the program only computes plane strains and stresses. ATENA 2D currently uses the Update Lagrangian formulation and supports the third level of nonlinearity (Červenka, Vladimír – Jendele, Libor – Červenka, Jan: ATENA Program documentation, Part 1 , Theory, Prague ,May 17,2000) . The material model of the program can incorporate material properties such as compression softening , tension softening , tension stiffening, fracture energy, etc.

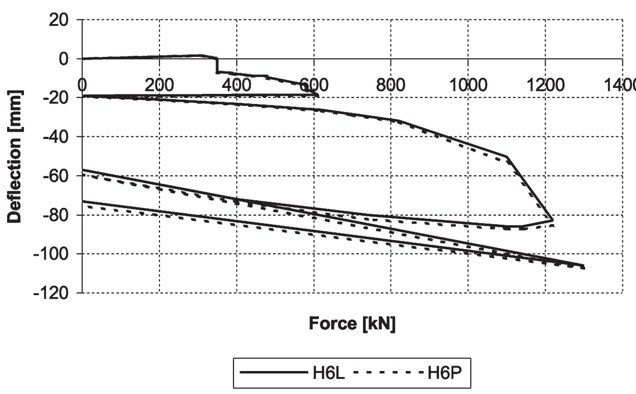

Fig. 9 *Force - deflection diagram*

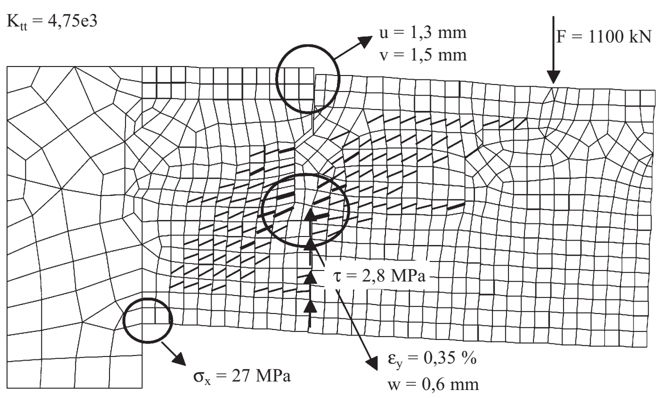

Fig. 11 *The best comparable model*

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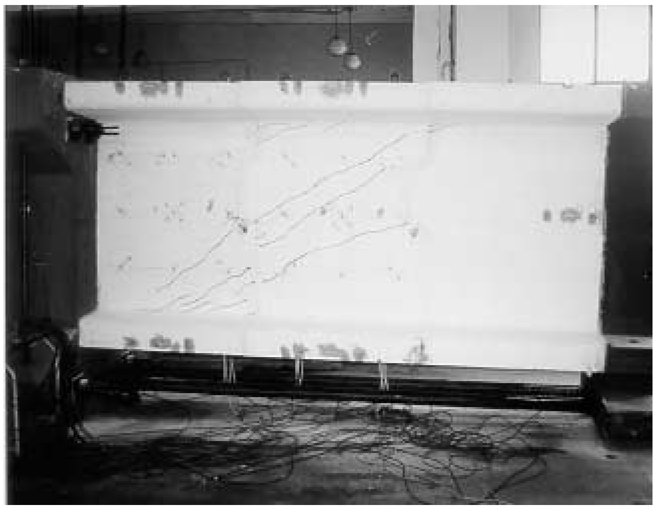

Fig. 12 *The best comparable model*

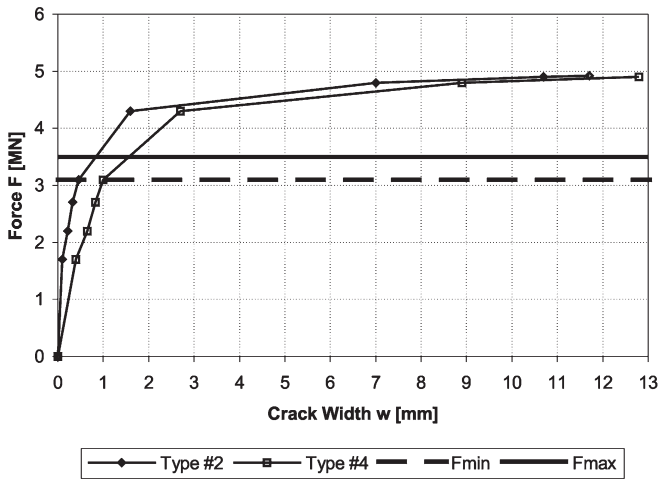

Fig. 13 *Force-crack width diagram*

Two models were created using the program. The geometry of the first model corresponded to the experimental model. This model was compared to the experimental results and then calibrated. The second's model geometry corresponded to the geometry of the bridge segment. This model used parameters adopted from the experimental model. The results, like computed crack inclination and crack width were compared to the crack pattern in the Kishwaukee bridge. To calibrate the calculations meant to create many models, each with different inputs. Parameter Ktt is one of two parameters which describe the so-called interface element. This interface element allows one to define differents friction between segments and opening of the joints in the case of tension stresses. Model type #1 represented a monolithic structure and model type #4, a structure with no friction between segments, and zero transfer of shear stresses across the joint. Types #2 and #3 lay between these boundary cases. The best comparable FEM model to the experiment is in Fig. 11. Each model was tested with four different values of parameter Ktt (four model types).

Conclusions

The aim of the project was the assessment of stresses in the damaged webs of segments of the southbound Kishwaukee river bridge. The paper concentrates only on the web of segment #2 (the second segment from the pier segment). The relation of the force versus crack width is in Fig. 13 for two boundary cases, K_{tt} – type #2 and type #4. Force Vmin is the shear force in the assumed web just after completion of the cantilever, and force Vmax is the shear force due to all permanent and vehicular loads.

The insertion of steel pins has improved the transfer of shear forces across the joints. The distribution of shear stresses is more uniform along the web's height, which corresponds with calculation type #2. The shift of shear properties due to the steel pins caused the current behavior of the structure to approach to the behavior of model type #2. Therefore in order, to make the shear crack wider after retrofit, greater shear force is needed than before the retrofit. This is also clear from Fig. 13, where it can be seen that the shear force Vmin causes wider shear cracks for model type #4 than Vmax for model type #2. This was also confirmed by the load test of the bridge (Halvoník, Jaroslav: Stress State Analysis of Southbound Kishwaukee Bridge, Habilitae Thesis, Bratislava, 2001.), when the cracks remained passive under imposed testing load.

The assessed strains and stresses were obtained from the calculation model[1], whose crack inclination and crack width are comparable to those recorded in the assumed web of the bridge see Fig. 14 and Fig. 15. The results from experimental test served for the calibration and adjustment of the calculation model. In Figs.13 and 14 extreme cases of behavior are shown, the so-called local extremes. For the stress assessment in the web of segment #2, average values of crack width and crack inclination were used. From the model type #4, the average crack width was 0.5mm and the average crack slope is 20°. This corresponds to the average stress in shear reinforcement σ_y = 360MPa.

[1] *Geometry of the model corresponds to the geometry of the bridge segment.*

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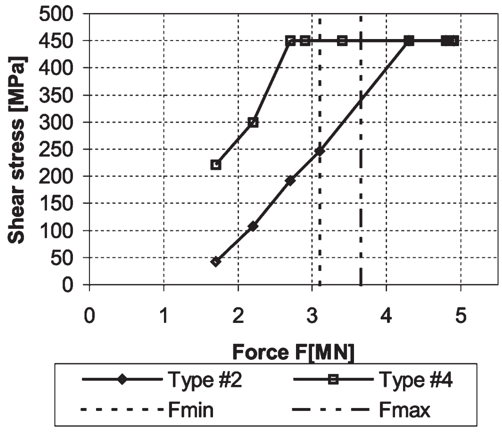

Fig. 14 *Force-stress diagram*

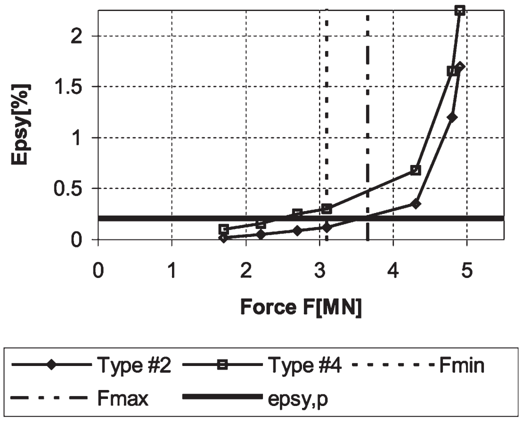

Fig. 15 *Force-strain diagram*

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