REPAIR DESIGN OF SHORT SPAN REINFORCED CONCRETE ROAD BRIDGE

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Summary: *This paper presents repair design for bridge over the Resnica River in Koceljeva municipality (Serbia). As part of repair strategy, visual inspection of damages and condition diagnostics of structural elements were conducted. In addition, the bridge was tested with trial static loads. Structural system of the existing bridge is constructed as a double overhanging slab with inner span of app. 4,5 m and cantilevers of approx. 1,0 m each. Main span is supported by transverse beams with variable cross-section which are constructed on top of the abutments. As part of repair design, the bridge was widened on each side with 1.0 m wide footways. Since the original structural design documents are missing, control calculations were conducted with Tower Radimpex Software (according to national codes). This analysis was followed by more sophisticated numerical modelling and analysis according to Eurocode with Abaqus Software.*

Keywords: *repair of bridge, static load test, bridge test, footways, bridge deflection*

1. INTRODUCTION

Bridge is situated on the local road over the Resnica River in the Koceljeva municipality. The existing road bridge is made of reinforced concrete, with an irregular foursquare base whose sides are 6,2 x 5,7 x 5,7 x 5,8 m. Road structure is reinforced concrete solid slab 22 cm thick, cast in situ. The existing width of the bridge, without footways, was 5,75-6,25 m, and the width of roadway is 5,4 m (measured perpendicular to the longitudinal axis of the bridge, which forms an angle of 71° with the river directly below the bridge). Static system for road structure is a double overhanging slab with inner span of app. 4,5 m and cantilevers of approx. 1,0 m each. In the support regions, the slab was constructed with depths of 75 cm on the left and 57 cm on the right bank, therefore forming transverse beams over the abutments. Abutments are partially made of masonry stone and of reinforced concrete. The average height from the upper level of the foundation to the lower level of bridge deck is app. 2,7 m. Abutments are supported with 40 cm wide foundation strip. There were no damages observed on the upstream side of

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the bridge, but on the downstream side a significant damage of the bridge deck was observed (hole 30-40 cm in diameter) as well as the separation of abutments from bridge deck on length of app. 75 cm. The abutment on the right upstream side was completely destroyed and was in a state of unstable equilibrium. In this condition, the bridge was used for one-way traffic but observed damage cast serious doubts on its load bearing capacity and reliability. The longitudinal and transverse views of the bridge are shown in Figure 1.

Figure 1. The transverse and longitudinal view of the bridge

2. BEHAVIOR OF THE BRIDGE UNDER TRIAL LOADING

Prior to testing with trial loads, a test program was prepared which included: size and layout of the test load by phases (in total, six load phases were defined); disposition of measurement locations and instruments; and organizational chart for testing. Only obtained results from the measurements and subsequent static and deformation analysis are presented within this paper without more detailed presentation of the test program. The bridge structure was tested with static load test using one 30980 kg heavy vehicle, which corresponds to the vehicle V300 from national codes, so the efficiency coefficient of the test load was 1,0 what is in accordance with standard SRPS U.M1.046 (National code that regulates testing procedures for road bridges). Figure 2 shows the disposition of measuring points for monitoring the deflection and schematic representation of the used vehicle.

Maximum measured deflection was 2,18 mm, which is a 1/206 of a bridge span. Residual deflection after unloading was 0,16 mm which is 7,34% of the maximum deflection. Numerical value of corresponding deflection that was obtained by the software package Tower (Radimpex, Belgrade) was 1,76 mm. Since the bridge span is rather small, both expected and measured values of deflection are of the magnitude that is difficult to observe without precise instruments and therefore they had no noticeable impact on bridge functionality or aesthetic appearance. In addition to measuring of deflection under test load, nondestructive testing of compressive strength of concrete was carried out with rebound hammer test. These tests were conducted on the bottom surface of the bridge between measuring points U4, U5 and U6 (Figure 2). Measured strength of concrete was evaluated at σ=33,73 MPa. Age of concrete was estimated at 20 years while concrete cover was rather small: 15 to 20 mm. Type and diameter of reinforcement was determined by locally removing concrete cover from the bottom surface of the bridge and it was determined that longitudinal bars were Φ16 mm with 15 to 20 cm spacing.

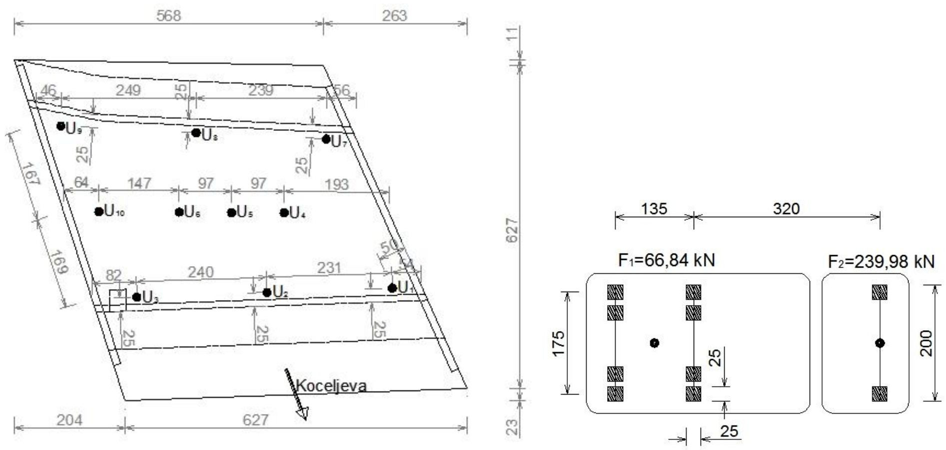

Figure 2. Disposition of measuring points and schematic representation of the used vehicle for test load

3. BRIDGE REPAIR STRATEGY

In order to widen the bridge with new footpaths it was planned to form reinforced concrete beams on both sides of the bridge (upstream and downstream). These beams are to be supported by new reinforced concrete columns. New columns are joined to the existing abutments while footpaths are separated from the main bridge deck. In order to repair damaged support downstream region of the left transverse beam, the project included its reconstruction and additional support by the new column. The joints between columns and abutments are constructed by installing reinforcement anchors. Repair design demanded a reconstruction and strengthening of the bottom surface of the bridge deck. This included cleaning of all concrete structural elements and reinforcements. The procedure was to be carried out by machines: picking, chiseling, wet sandblasting under high pressure, etc., until obtaining a clean, firm, "healthy" concrete surface. This procedure is to be carried out on the entire bottom surface of concrete deck surface. Upon completion of these works grouting of cracks wider than 0,5 mm in the concrete deck is planned while the smaller cracks 0,1-0,5 mm are to be filled with epoxy sealants. Surface damage of the reinforced concrete deck which have a depth up to 5 cm are performed in two layers: the basic and repair mortar. For determination of soil load capacity the geotechnical investigation were conducted on the existing foundations level and during the repair works the thickness of the foundation should be verified. As part of the repair works, riverbank protection should be carried out in the length of 50 m upstream and downstream to prevent future damages to abutments.

4. CONTROL STATIC CALCULATION OF THE BRIDGE ACCORDING TO NATIONAL STANDARDS

Load analysis included: permanent load, V300 vehicle traffic load, creep and shrinkage of concrete, actions on pedestrian barrier, temperature effects, braking and acceleration forces, snow load. Figure 3 shows a model of the extended bridge which is made with the Tower Software. Numerical results within this paper are given in a condensed form. Numerical analysis of the existing bridge showed a demand of A_{a1}= 6,83 cm^2 of reinforcements in the bottom zone of the span. During condition diagnostics, built-in reinforcement was estimated at Φ16mm with spacing of 15 to 20 cm what is 10,05 to 12,06 cm^2 i.e. greater than demanded reinforcements. Comparison of measured and calculated deflections obtained during this analysis also showed good agreements.

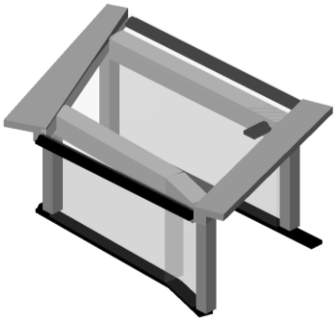

Figure 3. Numerical model of the widened bridge (Tower Software)

5. CONTROL STATIC CALCULATION OF THE BRIDGE ACCORDING TO EUROCODE

Within this analysis applied loads were analyzed as follows: self-weight was automatically included by the software, layers for pavement (wearing layer of asphalt 4 cm + asphalt protection layer 3 cm + waterproofing 1 cm =1,00+0,75+0,50=2,25 kN/m^2), footway curbs (assumed 0,5 kN/m), self-weight of pedestrian parapets (assumed 0,5 kN/m), assumed layers on footways (asphalt layer 3 cm + waterproofing 1 cm = 0,75+0,50 = 1,25 kN/m^2). Traffic load on the bridge is determined according to [1]. Since the width of the bridge traffic profile is 5,4 m, it was divided into one 3,0 m wide traffic lane and 2,4 m remaining traffic profile. Vertical traffic load according to Eurocode (Model LM1) is shown in Figure 4 (left). Concentrated load is distributed at an angle of 45° to the middle of concrete slab. The contact wheel surface is 0.4 m x 0.4 m while in middle plane of the concrete deck it is 0,7 m x 0.7 m. 300 kN vehicle was taken into account with dynamic coefficients. Recommended value of correction factors α_{Qi}=0,8; α_{qi}=1,0; α_{qr}=1,0 are used. The value of surface pressure within the 3 m wide traffic lane was 9kN/m^2, and on the remaining surface 2.5 kN/m^2. Vertical traffic load on the model LM2 (Figure 4 right) is 400 kN axle load, which is set in a designed position; in the middle of the traffic profile and in the center of the bridge. Load per one wheel is 200 kN and the contact wheel surface is 0,35 m x 0,6 m. Surface in the middle plane of the concrete deck is 0,65 m x 0,9 m. Horizontal traffic load (braking and acceleration forces) is applied in the longitudinal direction of the bridge, in the middle of the traffic lane and it can be positive and negative.

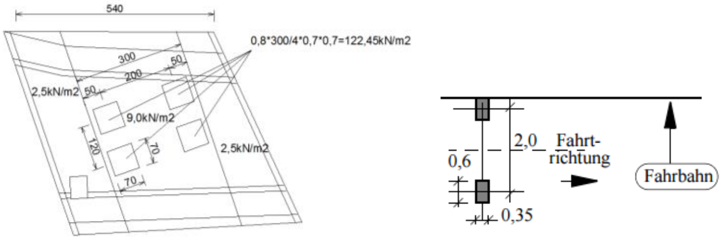

Figure 4. Traffic load according Model LM1 (left), LM2 (right)

Calculation of horizontal traffic load is given by the expression (1).

$$Q_{lk} = 0{,}6 * \alpha_{Q1} * (2 * Q_{1k}) + 0{,}1 * \alpha_{q1} * q_{1k} * w_1 * L$$
$$Q_{lk} = 0{,}6 * 0{,}8 * (2 * 300) + 0{,}1 * 1{,}0 * 9{,}0 * 3{,}0 * 5{,}8 = 303{,}66\,kN \tag{1}$$
$$q_{lk} = \frac{Q_{lk}}{L} = \frac{303{,}66}{5{,}8} = 52{,}37\,kN/m$$

Load value on footways is 5,0 kN/m^2, and the load on the pedestrian railing (horizontally and vertically 1,0 kN/m on top of the 1,0 m high railing). From accidental loads a 100 kN horizontal collision force on curbs on 0,5 m length and simultaneously vertical force of 0,75*0,8*300=180 kN are taken into account. For temperature effects, analysis takes into account uniform temperature component 33,6°C (approved data for the Koceljevo municipality on the basis of statistical analysis of temperatures) and recommended linear temperature component (upper side warmer 15°C or underside warmer 8°C) and recommended values according to [6]. Temperature influence is taken into account on the bridge and footways slabs only. Snow load was not taken into consideration with traffic load (model LM1 and LM2) [2]. Wind load and seismic influences were also excluded since it was assumed that they don't have a major contribution. Analysis of individual load cases and appropriate load combinations for ultimate limit state and serviceability were analyzed in accordance with [2]. Figure 5 shows the model of the widened bridge that was made in the software package Abaqus and figures 6 shows bending moment M_X for traffic load case of model LM2.

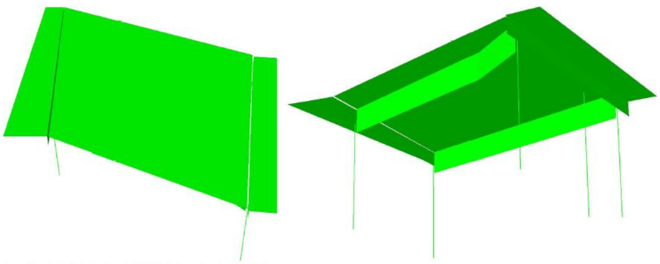

Figure 5. Model in Abaqus

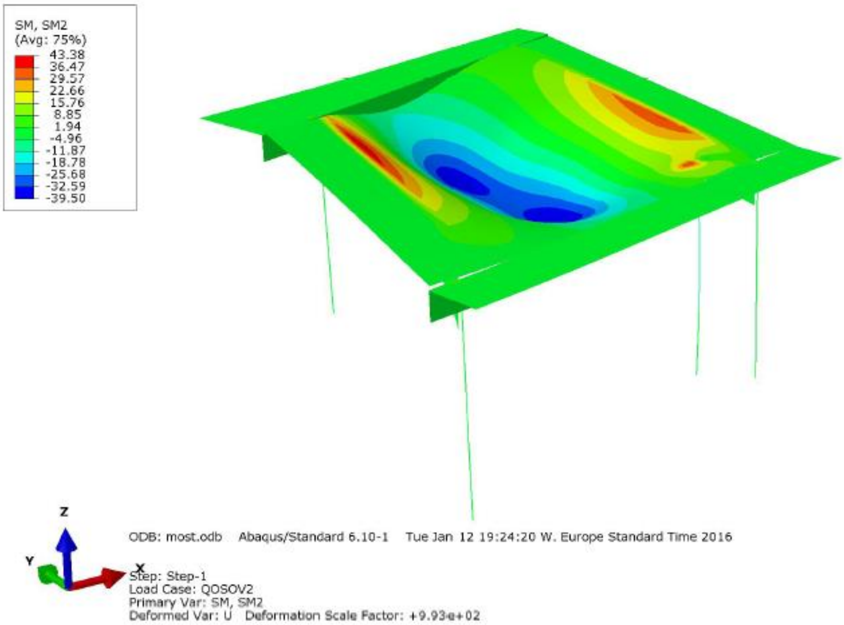

Figure 6. Bending moment M_x for load case LM2

For ultimate limit state the relevant combination of loads is established (1,35x permanent load+1,5xLM2) and after that calculation for required reinforcement in the span of concrete slab has been conducted according to the expressions (2).

$$
\begin{aligned}
& h = 22cm; c = 2cm \\
& d_1 = c + \frac{\phi}{2} = 2 + \frac{1{,}6}{2} = 2{,}8cm; d = h - d_1 = 22 - 2{,}8 = 19{,}2cm \\
& f_{cd} = \frac{25}{1{,}5} = 16{,}67\,MPa = 1{,}67\,kN/cm^2 \\
& f_{yd} = \frac{500}{1{,}15} = 434{,}78\,MPa = 43{,}48\,kN/cm^2 \\
& \mu_{Ed} = \frac{M_{Ed}}{b * d^2 * f_{cd}} = \frac{6359{,}5}{100 * 19{,}2^2 * 1{,}67} = 0{,}103 \\
& \xi = 0{,}138; \zeta = 0{,}944 \\
& A_{s1} = \frac{M_{Ed}}{\zeta * d * f_{yd}} = \frac{6359{,}5}{0{,}944 * 19{,}2 * 43{,}48} = 8{,}07\,cm^2
\end{aligned}
\tag{2}
$$

As seen above, reinforcement demand showed lower value than provided reinforcement. Stress control, crack width, minimum reinforcement and deflection control were conducted for serviceability limit state. According to [3] for concrete bridges it is necessary to check the concrete compressive stress which were also obtained from the numerical analysis. Code recommendations are that compressive stress do not exceed the value of k_1*f_{ck} while value of k_1 is recommended 0,6 (national standard BAB87 unlike Eurocode does not require the limitation of the stress during exploitation). It is assumed that concrete shows linear elastic properties for compressive stresses up to 0,4* f_{ck}. Maximum compressive stresses obtained during this analysis was, for the combination (1,0x permanent load +1,0x LM2), obtained by the expression (3),

$$\sigma = 0{,}66 + 5{,}71 = 6{,}36MPa < 0{,}6 * 25 = 15MPa \tag{3}$$

Calculation of the minimum reinforcement in the middle of the bridge for crack control is made according to the expression (4),

$$\begin{aligned}
A_{s\min} &= \frac{k_c * k * f_{ct,eff} * A_{ct}}{\sigma_s} \\
k_c &= 0{,}4; k = 1{,}0; f_{ct,eff} = f_{ctm} = 2{,}2Mpa \\
A_{ct} &= 11*100 = 1100cm^2; \sigma_s = 500Mpa \\
A_{s\min} &= \frac{0{,}4*1*2{,}2*1100}{500} = 1{,}94cm^2
\end{aligned} \tag{4}$$

proving that provided reinforcements is larger than the demanded for cracks control. Long term effects of creep also taken into account. The deflection is determined by the expression (5), [4].

$$\begin{aligned}
w(t,t_0) &= (1+\varphi(t,t_0)) * w(t_0) \\
w(t_0) &= 0{,}97\ mm
\end{aligned} \tag{5}$$

Maximum deflection value without concrete creep was 0,97 mm. In the expressions (6) a method of determination the linear creep coefficient for period of 20 years is shown, and then the corresponding value of deflection [5].

$$\begin{aligned}
h_0 &= \frac{2*A_c}{u} = \frac{2*100*22}{2*(100+22)} = 180{,}3\,mm \\
\varphi(t,t_0) &= \varphi_0 * \beta_c(t,t_0) \\
\varphi_0 &= \varphi_{RH} * \beta(f_{cm}) * \beta(t_0) \\
\varphi_{RH} &= 1 + \frac{1 - RH/100}{0{,}1*\sqrt[3]{h_0}} = 1 + \frac{1-70/100}{0{,}1*\sqrt[3]{180{,}3}} = 1{,}53 \\
f_{cm} &= f_{ck} + 8 = 25 + 8 = 33\,MPa \\
\beta(f_{cm}) &= \frac{16{,}8}{\sqrt{f_{cm}}} = \frac{16{,}8}{\sqrt{33}} = 2{,}93; \beta(t_0) = \frac{1}{(0{,}1 + t_0^{0,2})} \\
t_0 &= t_{0,T} * \left(\frac{9}{2 + t_{0,T}^{\ 1,2}} + 1\right)^{\alpha} = 10 * \left(\frac{9}{2+10^{1,2}} + 1\right)^1 = 15\,days
\end{aligned} \tag{6}$$

$$\beta(t_0) = \frac{1}{(0{,}1 + 15^{0{,}2})} = 0{,}55; \varphi_0 = 1{,}53 * 2{,}93 * 0{,}55 = 2{,}47$$

$$\beta_c(t, t_0) = (\frac{t - t_0}{\beta_H + t - t_0})^{0{,}3}$$

$$\beta_H = 1{,}5 * \left[1 + (0{,}012 * RH)^{18}\right] * h_0 + 250 = 1{,}5 * \left[1 + (0{,}012 * 70)^{18}\right] * 180{,}03 + 250 = 531{,}75$$

$$\beta_H = 531{,}75 \leq 1500$$

$$\beta_c(t, t_0) = (\frac{7300 - 15}{531{,}75 + 7300 - 15})^{0{,}3} = 0{,}98 \Rightarrow t = 20\ years$$

$$\varphi(t, t_0) = 2{,}47 * 0{,}98 = 2{,}42$$

$$w(t, t_0) = (1 + 2{,}42) * 0{,}97 = 3{,}31\, mm$$

As shown above, the value of the long term deflection in the middle of the bridge calculated according to Eurocode is approx. L/135 which is relatively large, however, since the bridge structure is at least 20 years old it is not expected that creep and shrinkage contribute significantly to the increase of the short term deflection which is approx. L/450.

6. CONCLUSIONS

Based on presented numerical analysis and results of testing it is concluded that this bridge can be upgraded for seriously damaged state to a state of reliable exploitation by implementing proposed repair strategy.
Combination of testing results and control static calculations for the bridge in the existing and repaired state showed that reinforcement demand is for existing state A_{a1}= 6,83 cm^2 (according to national codes) and A_{a1}= 8,07 cm^2 (according to Eurocode), therefore proving that provided reinforcement (Φ16 mm with 15 to 20 cm spacing) is sufficient for reliable exploitation with respect to bearing capacity demand. Validity of the numerical model was proven by comparison of measured and calculated deflection for the existing bridge.
The numerical model was further verified by conducting calculations with two widely used software packages. This model, slightly altered, was then used for repair and widening design of the bridge according to current national codes and according to Eurocode which proved that load bearing capacity of the bridge deck is sufficient for reliable use providing that obvious damages are repaired (hole in the deck and separation of the deck and the abutment on the downstream left side; abutment on the right upstream side).
Other repair works are less significant and can be categorized as regular maintenance. Finally it can be concluded that even in cases of bridges with significant damages an efficient repair and strengthening are possible provided that design strategy is complemented with sophisticated numerical analysis which is verified by measurements conducted under trial loading tests.

REFERENCES

[1] Eurocode 1: Einwirkungen auf Tragwerke – Teil 2: Verkehrslasten auf Brücken; Deutsche Fassung EN 1991-2:2003 + AC:2010
[2] Eurocode 0: Grundlagen der Tragwerksplanung; Deutsche Fassung EN 1990:2002 + A1:2005 + A1:2005/AC:2010
[3] Eurocode 2: Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken – Teil 2: Betonbrücken – Bemessungs- und Konstruktionsregeln; Deutsche Fassung EN 1992-2:2005 + AC:2008
[4] Stahlbeton 2006-04: Vereinfachte, wirklichkeitsanhe Ermittlung der Durchbiegung von Stahlbetonträgern unter Kriechbeanspruchung, Jörg Bockhold, Tobias Pfister
[5] Eurocode 2: Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken – Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau; Deutsche Fassung EN 1992-1-1:2004 + AC:2010
[6] Eurocode 1: Einwirkungen auf Tragwerke – Teil 1-5: Allgemeine Einwirkungen – Temperatureinwirkungen; Deutsche Fassung EN 1991-1-5:2003 + AC:2009

ПРОЈЕКАТ САНАЦИЈЕ ДРУМСКОГ АРМИРАНО-БЕТОНСКОГ МОСТА МАЊЕГ РАСПОНА

Резиме: *У раду је приказан пројекат санаицје моста преко реке Реснице у општини Коцељева. У склопу пројекта извршен је визуелни преглед и дијагностика стања конструкције моста. Поред тога, извршено је и испитивање моста пробним статичким оптерећењем. Конструктивни систем моста је плоча са два препуста при чему је унутрашњи распон приближно 4,5 m а препусти приближно 1,0 m. Распонска конструкција је ослоњена на попречне греде различитих геометријских карактеристика које се ослањају на врхове обалних стубова. У склопу пројекта санације планирано је и проширење моста пешачким стазама ширине 1,0 m. С обзиром да није била доступна оригинална техничка документација моста, спроведен је контролни статички прорачуни у програму Tower (према domaћим прописима). Након овог прорачуна, спроведено је и сложеније моделирање и анализа исте конструкције према Еврокоду помоћу софтверског пакета Abaqus.*

Кључне речи: *санација моста, пробно статичко оптерећење, испитивање моста, пешачке стазе, вертикалне деформације*