VERIFICATION AND IMPROVEMENT OF THE CONTINUOUS RIBBED BRIDGE DECK GRILLAGE MODEL BASED ON FIELD TESTING

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Original scientific paper

In this paper the deflections determined from short term static load testing on the approach viaducts of the bridge over the river Sava in Brod are used to evaluate different numerical grillage models. The approach viaduct decks are made up as partial-precast ribbed deck continuous over piers by casting a continuous concrete diaphragm between the ends of adjoining longitudinal girders. In practical calculations the longitudinal members are assumed to be fully continuous for traffic and composite loads using constant sectional properties along the longitudinal members although this assumption is not entirely correct. The aim of this paper is the verification and improvement of the grillage model of the concrete approach viaduct of the bridge over the river Sava in Brod utilizing field testing and numerical grillage analysis.

Keywords: continuous partial-precast ribbed deck, grillage model, static load testing

Provjera i unaprjeđenje roštiljnog modela kontinuiranog rebrastog mosta na temelju pokusnih ispitivanja

Izvorni znanstveni članak

U radu su korišteni progibi određeni na temelju pokusnog statičkog opterećenja na prilaznim vijaduktima mosta preko rijeke Save u Brodu da bi se ocijenili različiti numerički roštiljni modeli. Prilazni vijadukti su prema konstrukcijskoj vrsti polumontažni betonski rebrasti gredni mostovi. Kontinuitet rasponske konstrukcije iznad stupova ostvaren je povezivanjem uzdužnih nosača naknadno betoniranom betonskom dijafragmom. U statičkom smislu rasponske konstrukcije mogu se tretirati kao kontinuirani roštilji za dodatno stalno i prometno opterećenje. U praktičnim je proračunima uobičajeno kod takvog tipa rasponskih konstrukcija pretpostaviti potpuni kontinuitet uzdužnih greda konstrukcije uz pretpostavku nepromjenjivih mehaničkih karakteristika u uzdužnom smjeru, premda takva pretpostavka nije u potpunosti točna. Cilj ovog rada je provjera i unaprjeđenje roštiljnih modela betonskih rebrastih prilaznih vijadukata mosta preko Save u Brodu korištenjem rezultata pokusnih ispitivanja i rezultata numeričkih proračuna na roštiljnim modelima.

Ključne riječi: kontinuirani polumontažni rebrasti gredni most, pokusno statičko opterećenje, roštiljni model

1 Introduction

In the last 60 years precast prestressed concrete bridges became one of the key solutions in bridging short and medium span range, 20÷50 meters [1].

According to available data [2] 80 % of bridges on Croatian' highways are bridges with small and medium span length and among them 90 % are ribbed bridge deck structures. Similar data can be found for bridges on highways in Italy, France, Holland and Germany [2]. The data of the research conducted in the USA, Canada and Japan, represented in paper [3] show that continuous precast prestressed concrete bridges are the most commonly used bridge type on interstates and high volume urban highways.

The ribbed bridge decks composed of longitudinal beams (main girders), transverse beams (diaphragms) and deck slab provided a convenient and cost effective solution [4] for multi span beam bridges composed by a series of simply supported beams of uniform length but it may be also adapted to continuous bridge structures.

Partially precast concrete bridge deck made up of precast prestressed main girders, cast-in-place reinforced concrete diaphragms over supports and reinforced concrete deck slab are the most usual deck structure built in past two decades in Croatia for small and medium span length bridges [5, 6]. To provide continuity of deck structure in longitudinal direction, the main girders can be connected together by cast in place diaphragm over support as it is shown in Fig. 1. In this case the diaphragm can be treated as continuity joint between main girders.

The main girders joined together at the end by continuity joint are usually modelled as fully continuous for traffic and composite loads using constant sectional

properties along the longitudinal members [7, 8] although this assumption is not entirely correct.
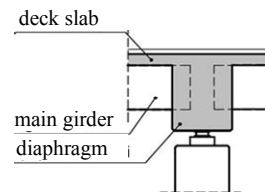

Figure 1 The cross section of continuity joint

In this paper the results of short term static load testing on the bridge over the river Sava in Brod were used to check and improve a numerical grillage model of concrete ribbed bridge deck of the bridge. The improvement of the model implies reduction of main girder stiffness in the connection zone (left and right of the continuity joint).

2 The bridge over the river Sava in Brod
2.1 General information

The bridge is composed of two main structural deck systems: the main steel truss bridge over the river Sava and the concrete approach viaducts [9, 10]. Only the continuous concrete deck structures of the approach viaduct (the south viaduct and the central viaduct) will be analysed in this paper. The south viaduct is a two span structure (2 × 30,00 m) and the central viaduct is a six span structure (23,15 m + 4 × 28,50 m + 23,15 m). The bridge deck cross section shown in Fig. 2 is the same for all approach viaducts. The detail of the continuity joint

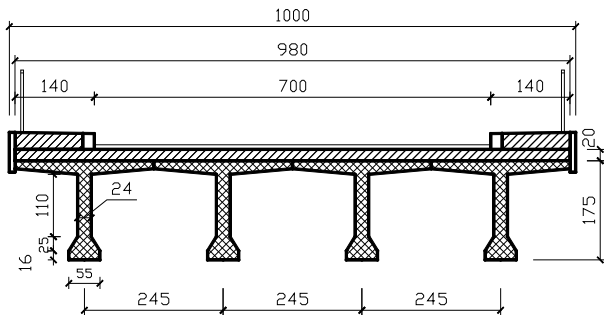between main girders over the mid support is shown in Fig. 3.

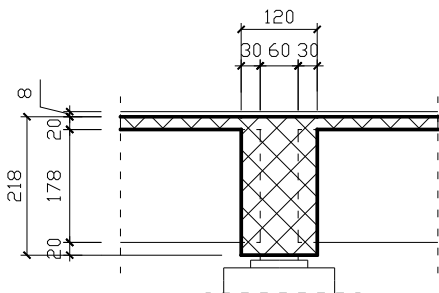

Figure 2 The cross-section of the approach viaducts

Figure 3 The detail of the continuitiy joint over the mid supports

2.2
Bridge load testing

The results of a field short term static load testing on the concrete part of the bridge over the river Sava in Brod were used. The bridge load testing was carried out by using heavy trucks. Weight of the trucks and their positions in field tests simulate the design traffic load on the bridge. In Figs. 4 and 5 the truck position in the middle of the bridge span is shown. The trucks scheme and axle masses are shown in Fig. 6 and Tab. 1.
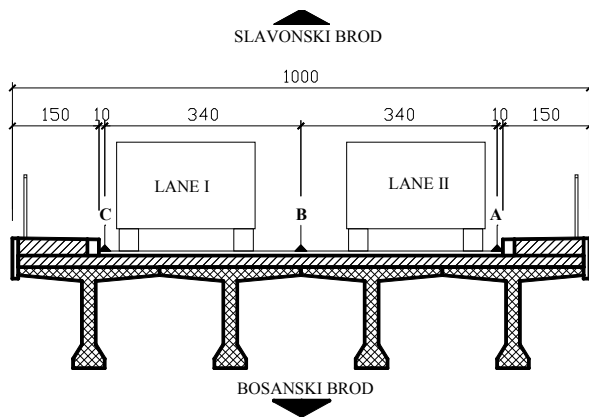

Figure 4 The longitudinal measurement lines A, B and C and the truck position in the bridge cross-section

The vertical displacements were measured by using precise levelling geodetic method [10] in measurement points located at the intersections of the longitudinal and transversal measurement lines, i.e. measurement points 1A, 1B, 1C, 2A, 2B, 2C,…13A, 13B, 13C. The position of longitudinal measurement lines is shown in Figs. 4 and 5. The transversal measurement lines are placed in the middle of the spans and over supports according to Figs. 7 and 8.
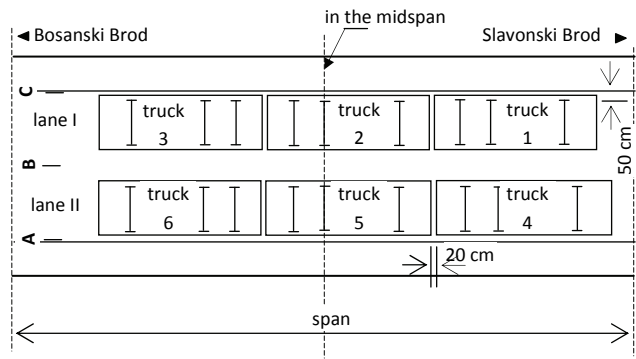

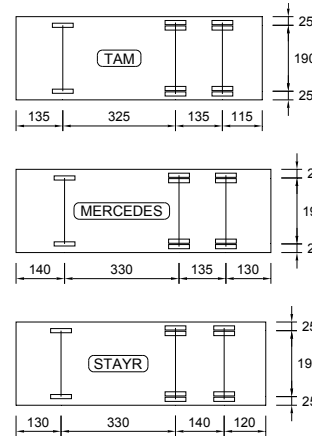**Figure 5** The plan view of truck position in the middle of the span

Figure 6 The truck schemes

Table 1 The trucks axle masses (in kg)

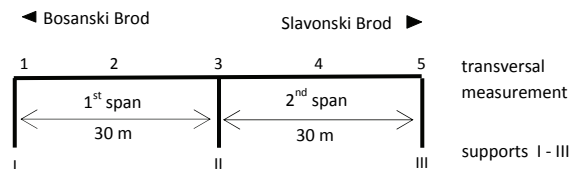| Truck number	Mass of front axle	Total mass of two back axles	Type of truck
1	7140	23260	TAM
2	7580	23740	TAM
3	7180	23420	TAM
4	7260	23780	STAYR
5	7020	24580	MERCEDES
6	7580	24780	TAM

Figure 7 The transversal measuremet lines 1-5 in the longitudinal view of the south viaduct
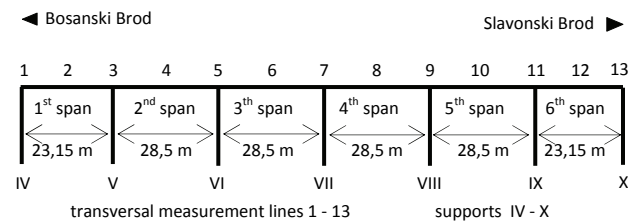

Figure 8 The transversal measurement lines 1-13 in the longitudinal view of the central viaduct

The vertical displacements measured in load phases displayed in Tabs. 2 and 3 were used. From this load phases the lifting of the unloaded spans nearest to the loaded one was possible to be measured. The other phases that were conducted in static load testing were not analyzed in this paper.

Table 2 Load phases on the south viaduct

Load phase	Loading schemes		
	1st span	2nd span	
I	▭		▭ three trucks in lane I
II	▭		
III		▭	▭▭ six trucks

Table 3 Load phases on the central viaduct

Load phase	Loading schemes					
	1st span	2nd span	3rd span	4th span	5th span	6th span
I	▭					
II		▭				
III			▭			
IV				▭		
V					▭	
VI						▭

▭ six trucks

2.3
The results of field load testing

In each load phase the vertical displacements were measured only in the loaded span and the adjacent unloaded span(s) and over their nearest supports.

The measured deflection v_m in the middle of the span is calculated by using the measured vertical displacement in the middle of the span and the measured vertical displacement at the nearest supports [10]. In Tables 4 and 5 the mean values of measured deflections in each transversal measurement line are shown.

Table 4 Measured deflection v_m on the south viaduct (in mm)

Transversal measurement line	Load phase		
	I	II	III
2	3,12	6,6	−2,65
4	−0,92	−2,05	7,28

Table 5 Measured deflection v_m on the central viaduct (in mm)

Transversal measurement line	Load phase					
	I	II	III	IV	V	VI
2	3,02	−1,5	-	-	-	-
4	−1,02	3,90	−1,53	-	-	-
6	-	−1,45	4,43	−1,43	-	-
8	-	-	−1,37	4,13	−1,52	-
10	-	-	-	−1,43	4,23	−0,95
12	-	-	-	-	−1,42	3,17

3
Basic numerical models for the approach viaducts

The central viaduct deck is modelled as equivalent grillage over six spans and the south viaduct deck is modelled as equivalent grillage over two spans (Figs. 9 and 10). The models consist of longitudinal beam elements which represent main girders and transversal beam elements which represent diaphragms over supports and deck slab. In this model the assumption of full continuity in longitudinal direction for traffic and composite loads using constant sectional properties is adopted.

Sectional properties of grillage elements are determined according to the elements dimensions in design documentation [9]. Young's modulus is $E = 3,4 \times 10^7$ kN/m^2 for main girders and $E = 3,15 \times 10^7$ kN/m^2 for diaphragms and deck slab according to [10].
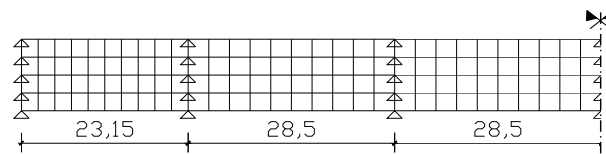
Figure 9 Grillage model of the central viaduct (a left half)
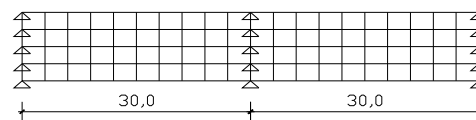
Figure 10 Grillage model of the south viaduct

3.1
Calculated deflections

Measured deflections have to be compared with the calculated ones. In Tables 6 and 7, the mean calculated deflections in the middle of each span for different load phases on the central and south viaduct are shown.

Table 6 Calculated deflection v_c on the central viaduct (in mm)

Transversal measurement line	Load phase					
	I	II	III	IV	V	VI
2	4,09	−1,92	-	-	-	-
4	−1,75	5,75	−2,01	-	-	-
6	-	−2,06	5,77	−2,02	-	-
8	-	-	−2,06	5,77	−2,01	-
10	-	-	-	−2,05	5,72	−1,76
12	-	-	-	-	−1,97	4,12

Table 7 Calculated deflection v_c on the south viaduct (in mm)

Transversal measurement line	Load phase		
	I	II	III
2	4,69	9,47	−3,85
4	−1,93	−3,91	9,53

4
Comparison of measured and calculated deflection on basic models

Comparison of calculated and measured deflection is conducted for 6 load phases on the central viaduct and for 3 load phases on the south viaduct. The comparison is based on the ratio of measured and calculated deflection in the loaded span and adjacent, unloaded spans (or span) for each load phase. The ratio of measured and calculated deflection in the loaded span should be equal to the ratio of measured and calculated deflection in adjacent unloaded spans assuming the same behaviour of the bridge and its numerical model.

Deviations of described ratios can be used as the evaluation of the numerical model behaviour in relation to the behaviour of the real bridge.

To simplify the comparison the deflections in each load phase were normalized to the maximum deflection value. The normalization was conducted for the measured as well as for the calculated deflections. In Figs. 11 to 19 the conducted comparison is shown.
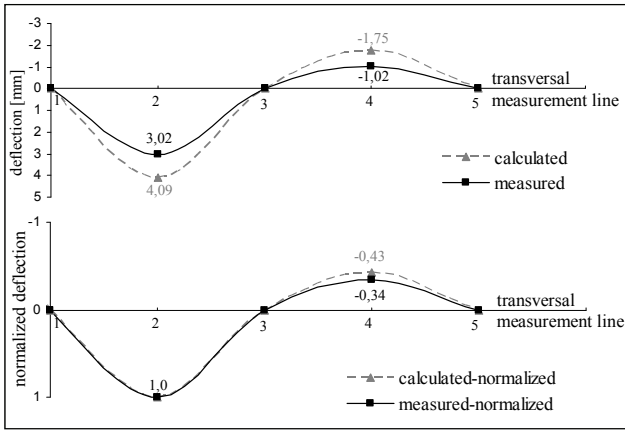
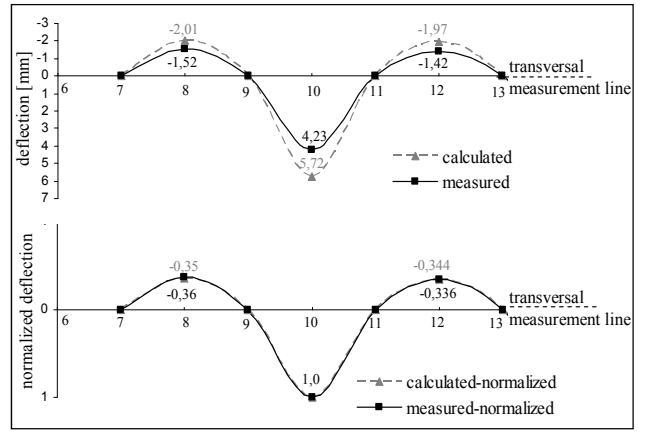**Figure 11** Deflections on the central viaduct for load phase I

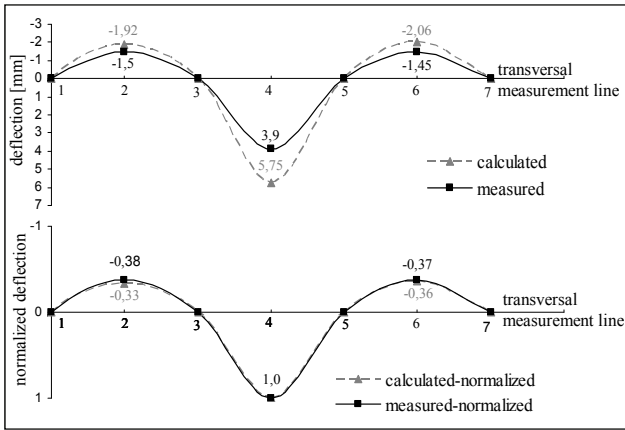

Figure 15 Deflections on the central viaduct for load phase V

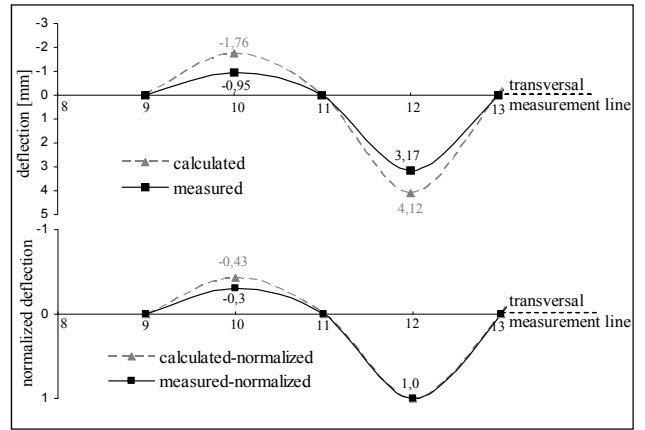**Figure 12** Deflections on the central viaduct for load phase II

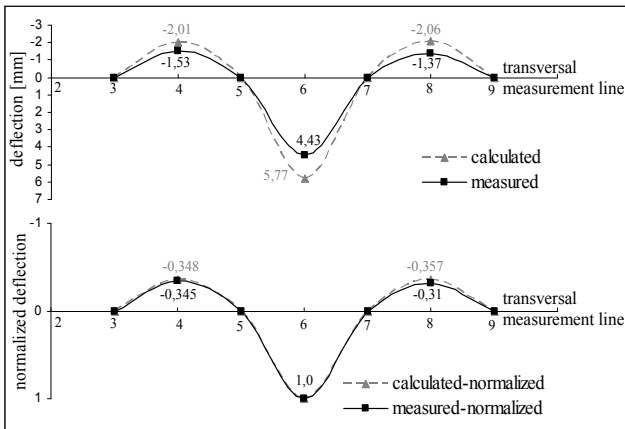**Figure 16** Deflections on the central viaduct for load phase VI

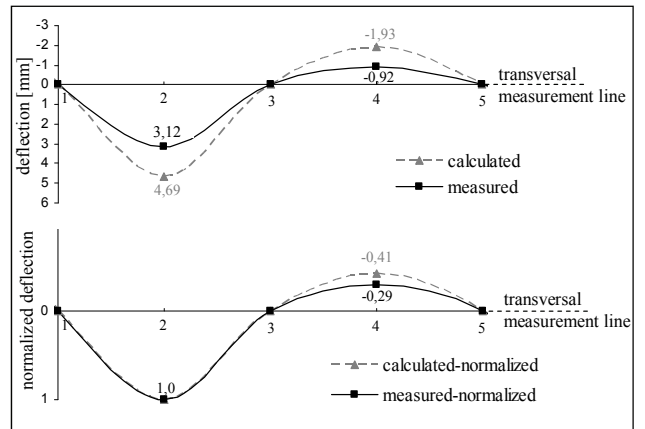**Figure 13** Deflections on the central viaduct for load phase III

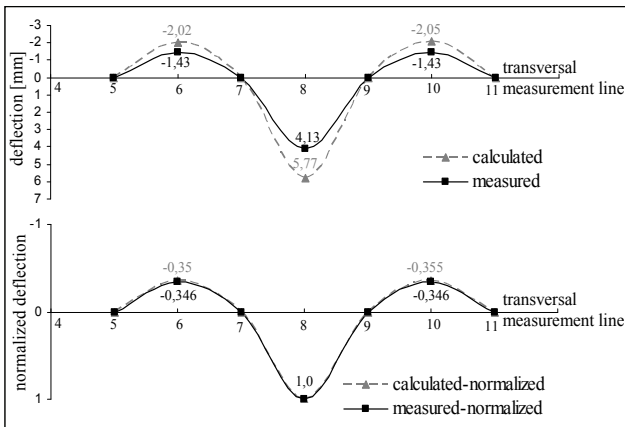**Figure 17** Deflections on the south viaduct for load phase I

Figure 14 Deflections on the central viaduct for load phase IV
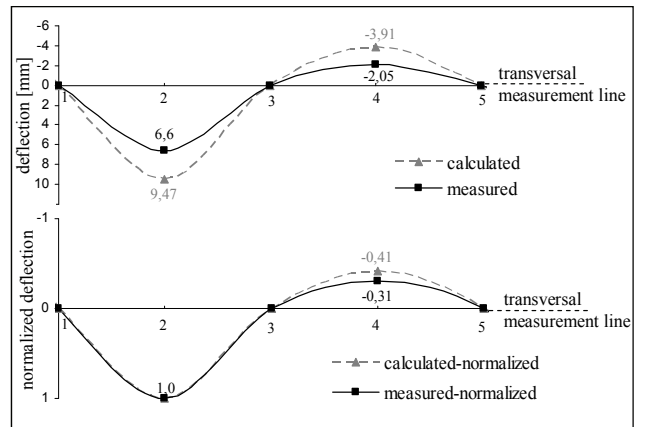

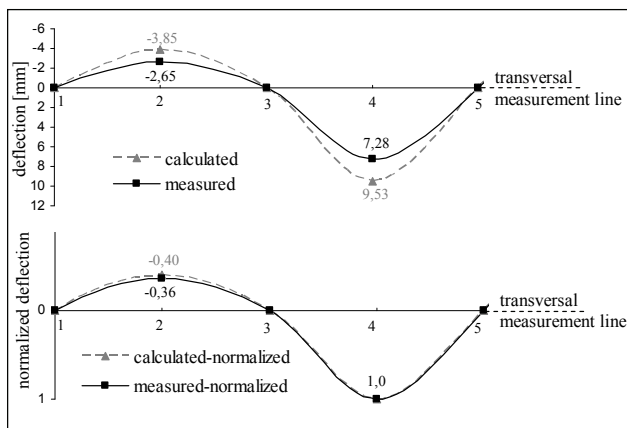**Figure 18** Deflections on the south viaduct for load phase II

Figure 19 Deflections on the south viaduct for load phase III

Deviation of normalized values of the measured deflection in comparison to the normalized values of the calculated deflection in unloaded span indicates a potential change in the continuity of the longitudinal members which are modelled as fully continuous using constant sectional properties along the members in numerical model for traffic and composite loads.

Considering that the main girders are precast prestressed concrete beams which are fully prestressed for sagging moment, there is no variation in girder's stiffness in the sagging moment area. This assumption leads to the conclusion that the main girders have different stiffness in a part of the hogging moment zone in which cracks are expected, i.e. in the vicinity of the continuity joints. In the zone of hogging moments the main girders are reinforced with non-prestressed steel. There is no prestressing in the hogging moment area.

The ratio of the normalized values of measured deflection $v_{m,n}$ in relation to the normalized value of the calculated deflection $v_{c,n}$ in unloaded span is defined as δ according to Eq. (1).

$$\delta = \frac{v_{m,n}}{v_{c,n}}. \tag{1}$$

Values of the ratios δ for both viaducts are shown in Tables 8 and 9.

Table 8 The ratios δ for the central viaduct

Transversal measurement line	Load phase					
	I	II	III	IV	V	VI
2	-	1,15	-	-	-	-
4	0,79	-	0,99	-	-	-
6	-	1,04	-	0,99	-	-
8	-	-	0,87	-	1,02	-
10	-	-	-	0,97	-	0,70
12	-	-	-		0,97	-

Table 9 The ratios δ for the south viaduct

Transversal measurement line	Load phase		
	I	II	III
2	-	-	0,90
4	0,71	0,75	-

The mean value $\bar{\delta}$ of the calculated ratios δ for both viaducts for the basic numerical model is 0,91.

5
Modified numerical models

In the paper [11] the length of the longitudinal element where the reducing of element stiffness as a result of upper zone cracking above the supports could occur is determined.
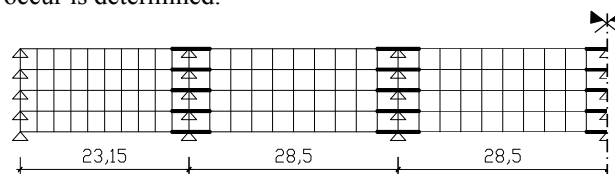

Figure 20 Modified grillage model of the central viaduct (a left half)

A length equal to 10 % of the span is taken in further analyses. This length is approximately corresponding to the zone in which the mean tensile strength of concrete has been exceeded according to EC2 [12]. It is also a length of one beam element in longitudinal direction.

Varying the stiffness of the longitudinal beam elements near mid supports is used to improve the basic numerical model.
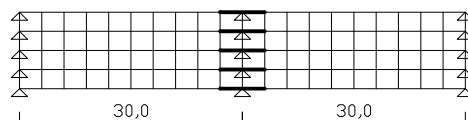

Figure 21 Modified grillage model of the south viaduct

The modified numerical models of the central and the south viaduct were modelled by adopting the assumption that the beam elements near mid supports have reduced stiffness. In Figs. 20 and 21 the beam elements with reduced stiffness are set in bold.
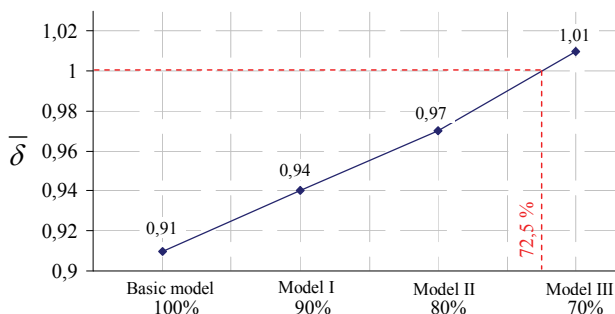

Figure 22 The value $\bar{\delta}$ in relation to different numerical models

In order to improve the numerical model three different models were made. In Model I the beam elements near mid supports has 90 % of stiffness of the same elements in the basic model, in Model II 80 % and in model III 70%.

As in the previous case, for the basic numerical model, the comparisons of the measured and the calculated deflection were conducted and the mean values $\bar{\delta}$ of the calculated ratios δ are determined. For Model I $\bar{\delta}$ = 0,94, for Model II $\bar{\delta}$ = 0,97 and for Model III $\bar{\delta}$ = 1,01.

The values $\bar{\delta}$ for all analyzed models are shown in Fig. 22. It can be concluded that Model III can be adopted as an improved model that accurately enough describes the real behaviour of the tested bridge.

6
Comparison of bending moments

Reduction of the stiffness of some beam elements in the improved Model III in relation to the basic model results in a change of bending moments of the main girders. In paper [11] the comparison of bending moments of main girders due to traffic load (V600) [13] and composite loads for the basic model and Model III was conducted.

Modified Model III in relation to the basic model results in an increase in maximum sagging moment of 6 % and decrease in maximum hogging moment of 8 % for the most unfavourable load combination, respectively.

The difference in maximum sagging moment for the two analysed models is only 2 % for the most unfavourable load combination for prestressing design (including self weight of the main girder).

7
Conclusion

The results of field load testing conducted on the bridge over the river Sava in Brod whose approach viaducts decks are partial-precast ribbed deck continuous over the piers by casting a continuous joint are used to evaluate different numerical grillage models.

During the short term static load testing vertical displacements were measured in the middle of the spans and over supports under different loading.

In order to compare the measured and the calculated deflection the basic numerical grillage models of the central and the south viaduct were modelled.

Ratio δ of normalized values of the measured deflection in comparison to the normalized values of the calculated deflection in unloaded span for each load phase was used to verify the accuracy of the basic numerical model. The mean value $\bar{\delta}$ of the calculated ratios δ for both viaduct for the basic numerical model is 0,91 and indicates that the behaviour of the structure is different from the behaviour estimated by basic numerical model.

This fact can be attributed to the simplified modelling, i.e. to the assumption of full continuity and constant sectional properties along the longitudinal beam elements.

The conducted analyses determined that the tensile stresses exceeding the mean tensile strength of concrete of the main girders can be expected to be up to a distance of $0,1L$ from the mid supports, therefore, the modified grillage models with reduced stiffness of some longitudinal beam elements were modelled. Three modified models with different stiffness reduction of the beam elements near mid supports were used.

For all modified models a comparison of measured and calculated deflection was made in the same way as for the basic model, i.e. the mean value $\bar{\delta}$ for each model was calculated. From conducted analyses it can be concluded that the modified Model III with the reduction of stiffness of 30 % in relation to the basic model accurately enough describes the behaviour of the real bridge under test loading.

Change in stiffness can significantly affect the distribution of internal forces in the structure. This fact is especially important for verifying the serviceability limit state and for tendons design in the prestressed bridge structures. In the case of the analysed viaducts of the bridge over the river Sava in Brod the increase in sagging moment of 2 % is a negligible value for tendon design, but the decrease of 8 % in hogging moment enables a reduction of reinforced steel above the mid supports.

This paper shows that the results of load testing which have been regularly carried out in Croatia after the new bridge was constructed can be used to verify and improve the numerical model used in bridge design.

8
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