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FIELD TESTS AND NUMERICAL ANALYSIS OF LATERAL LOAD DISTRIBUTION FOR RC ROAD BRIDGES

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Abstract: *This paper presents a comparative analysis of different strategies for numerical modelling of reinforced concrete semi-prefabricated bridges ("bulb-tee" bridges) and their experimental testing under trial traffic loads. Presented numerical strategies include: classical hand calculation methods, simplified 3D finite element method (FEM) and quasi 3D FEM calculations. Presented numerical results were verified against the results of testing of two similar bulb-tee bridges under trial traffic loads.*

Key words: *RC bridge, numerical simulation, testing, trial traffic load*

1. Introduction

Semi-prefabricated reinforced concrete bridges are widely used, especially for small to medium spans from 15 to 40 meters [5]. From the economic stand point, their cross sections represent some of the most rational structural designs particularly when positive bending moments are concerned. These bridges are often constructed from several identical prefabricated RC beams, placed next to each other with or without joining their upper flanges. The variation with joined upper flanges is additionally convenient since it requires no scaffolding for concreting of the roadway slab which, when it cures, connects and unifies all structural elements in one structural system. Common name in various contemporary technical documents for these bridges is "bulb-tee bridges".

Bearing capacity of these bridges in longitudinal direction is ensured by the main longitudinal girders, which are interconnected by the roadway slab. The slab transfers traffic loads to the girders and it also serves as a part of their upper flange. By acting in unisons with girder, the main, now practically composite, longitudinal girder has its centroid moved closer to the upper surface of the slab. This causes an increase in strains and stresses that are much higher in the lower section of the girder when compared to the upper sections [9].

Cross sectional forces, bending moments M_x, M_y and M_{xy}, and other load effects can be determined approximatively by means of influence surfaces (Pucher, Homberg-Ropers and other) or by means of tables defined for specific loads like for example DIN 1072 codes (Rüsch). Both procedures require effective width of the load transfer to be determined in longitudinal and in lateral direction. Influence surfaces are determined for

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two limit boundary conditions: for slabs that are freely supported on their edges and for slabs that are fixed within the webs of the main girders. The actual degree of the fixed supports has to be determined on the basis of torsional rigidity of the main girders.

For determination of forces in longitudinal cross sections of the main girders, a lateral load distribution has to be taken into account that is considerably affected by the width of the bridge and the number of loaded lanes [3]. Lateral load distribution is determined according to the line of the distribution which is different for each longitudinal girder, as described, for example in the Courbons method of eccentric loads. The influence line of the lateral load distribution has the shape of the deformation line of the bridge cross section loaded by unity force placed above the considered longitudinal girder [6]. Due to its simplicity with respect to numerical efforts this method was considered appropriate for a long time in our engineering practice. With the increase of the computer processors strength and sophistication of the software, the applicability of this method was brought to question. New finite element software like Ansys®, Abaqus® [1], [2], Tower® [8] and similar, have unlimited, in practical engineering sense, abilities to make structural simulations and therefore can be used for structural design instead of highly approximate procedures that were used before. These numerical strategies can be applied both to new bridges such as described in this paper and to existing bridges for strengthening and/or deconstruction [4].

On the other hand, in order to capture complex load distributions and interactions of all structural elements within a structure, like in semi-prefabricated bridge, a realistic 3D structural model has to be created and it has to utilize sophisticated numerical strategies in order to achieve this goal. This requires a lot of work hours invested by highly trained engineers and it yields numerical models with huge output files that often hinder a realistic structural analysis of the problem. This results in simplifications of the 3D models that visually look just like the real structure but lack the complexity of the load transfer that a real structure has. As a final result we often have a "nice looking" 3D models that provide less accurate results than approximate calculation strategies we used before.

Having in mind all of this, an effort has been made within this paper to present one practical numerical strategy that relies on older simplistic methods but utilizes modern software while providing outputs that are accurate enough and can be presented in a way a structural engineer can use them for serious structural design of semi-prefabricated concrete bridges. For comparison, a simpler 3D model has been created and all the results are verified against the experimental results obtained from two similar real road bridges that were tested under trial loads prior to their official introduction to service.

2. Description of the Structure and Applied Loads

This paper is concentrated on reinforced concrete bulb-tee bridge [7]. The structural system of the bridge is a simple span beam with a span of 14 m. In lateral direction, the bridge consists of 5 prefabricated "T" longitudinal girders with a 20 cm thick roadway slab constructed above. Lateral girders that would connect individual longitudinal girders were not applied so the torsional stiffness off the bridge is defined by the interaction of the roadway slab and longitudinal girders alone. Lateral distance between longitudinal girders axes is 151 cm and their height is 90 cm, as shown in the Figure 1 and 2.

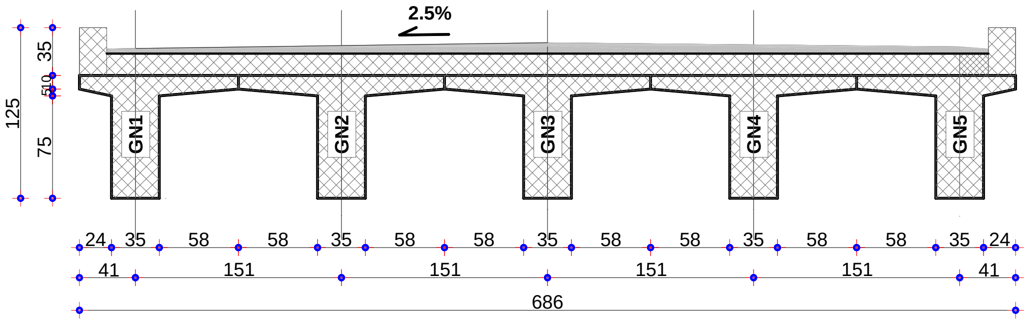

Fig. 1. *Cross section of the bridge - drawing*

Fig. 2. *Bottom view of the tested bridge*

The bridge trial load was a standard truck with one axle in front and two axels in the rear. The truck was loaded with gravel and measured.

Load intensity and distribution was determined by measurement performed for the front and the rear axle and the total weight of the truck:

- front axle (2 wheels): 6120 kg,
- rear axle (8 wheels): 24680 kg,
- total weight: 30800 kg.

The wheel surface was measured at an average of 48×30 cm. The longitudinal shape of the bridge with the truck positioned on it is showed in the Figure 3 and 4.

Fig. 3. *Photo of the Bridge 1 and the Bridge 2 with trail load, longitudinal view*

In order to investigate realistic traffic situations and lateral load distributions, a truck was positioned near the curb of the bridge. The measurements were conducted for two longitudinal positions of the truck, when rear axle is positioned above the cross sections at L/4 and at L/2 of the bridge span. Figure 4 shows a truck with a rear axle positioned above the cross section at L/4 of the span.

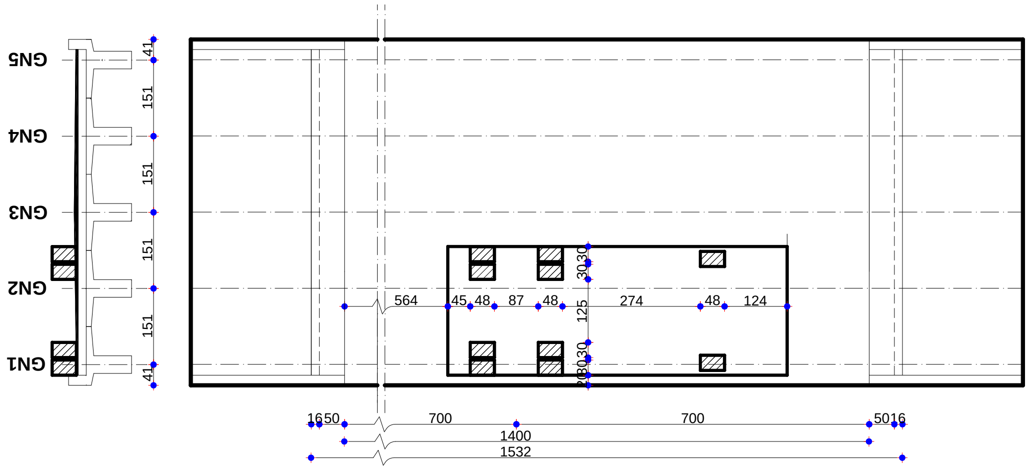

Fig. 4. *Schematic top view of the bridge and its loading*

3. Representative Stresses Obtained from Numerical Analysis and from Testing of the Structure

3.1. Calculation according to Courbons Method of Eccentric Loads

According to this method, when the ratio of the span length “L” and the distance of the edge girders “b”, is L/b≥2, Figure 5, it is assumed that the bridge is stiff in lateral direction and that its deformation line is straight so that the load distribution for each longitudinal girder can be determined as analogue to stresses of the plane cross section loaded with longitudinal force and bending moment according to expression (1):

$$\sigma = \frac{F}{A} \pm \frac{M\xi}{I} \quad (1)$$

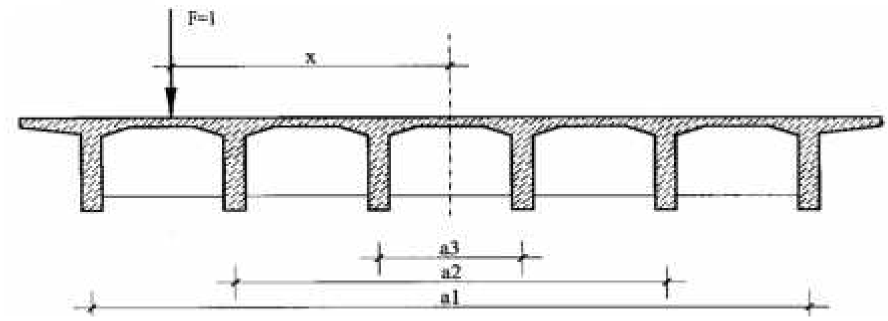

Fig. 5. *Eccentric loads method [3]*

When following is introduced:

$$\sigma = \frac{S_i}{A_i},\quad A = \sum_{i=1}^{n} I_i,\quad I = 2\sum I_i\left(\frac{a_i}{2}\right)^2,\quad M = F \cdot x \quad i \quad \xi = \frac{a_i}{2} \tag{2}$$

the expression (1) can be written as follows:

$$\frac{S_i}{I_i} = \frac{F}{\sum I_i} \pm \frac{F \cdot x}{2\sum I_i\left(\frac{a_i}{2}\right)^2}\frac{a_i}{2},\ \text{or}\ S_i = F\left(\frac{I_i}{\sum I_i} \pm \frac{I_i \cdot a_i \cdot x}{\sum I_i \cdot a_i^2}\right). \tag{3}$$

For $F=1$ and I_i=*const*, values of the influence line are obtained:

$$\eta_i = 1\left(\frac{1}{n} \pm \frac{a_i \cdot x_i}{\sum a_i^2}\right), \tag{4}$$

where *n* represent a number of longitudinal main girders.

According to [4], following shape of the influence line for the longitudinal main girder GN2 are obtained (Figure 6)

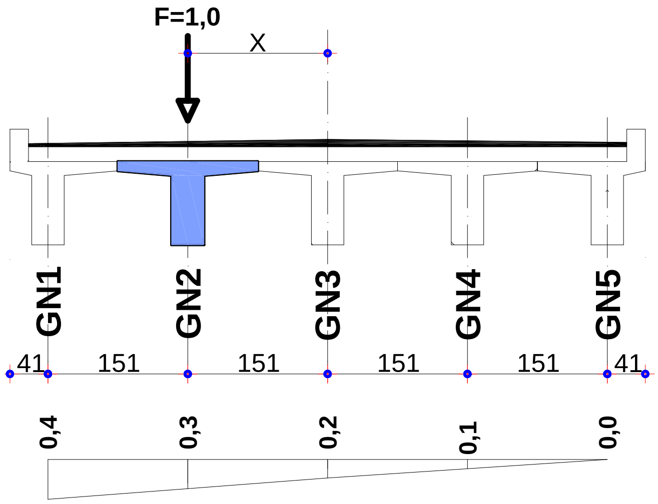

Fig. 6. *Influence line for lateral load distribution for the girder GN2*

Based on the obtained influence line, it can be determined that the girder GN2 has the following portion of the traffic load as shown in the Table 1.

Portion of the traffic load for the girder GN2 Table 1

Longitudinal girder	Front axle [kN]	Rear axle [kN]		**Total [kN]**
		1st row of wheels	2nd row of wheels	
GN2	20,33	40,66	40,66	**101,65**

Relative to the load position, bending moment for this girder is determined on the basis of the influence line for the bending moment for the filed cross section and portion of the load determined by the Courbon's method.

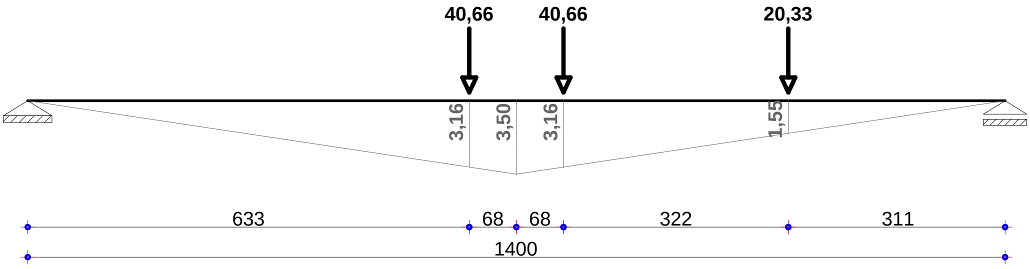

Fig. 7. *Influence line for the bending moment in the field cross section of the longitudinal girder GN2*

$$M=40,66ò(3,16+3,16)+20,33ò1,55=288,48kNm \quad (5)$$

Stress values on the bottom and the top of this girder obtained for this bending moment, Figure 7, in the girder GN2 are given in the Figure 8.

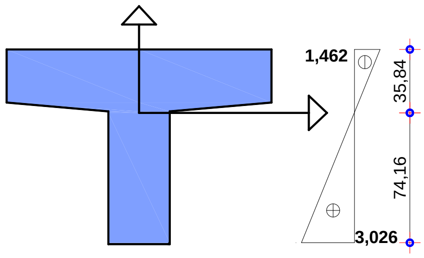

Fig. 8. *Stress diagram for the middle cross section of the GN2 [MPa]*

3.2. Stresses obtained with 3D Finite Element Model with Simplified Material Models and Boundary Conditions

Numerical model of the bridge structure and the traffic load presented in this paper were prepared in the ABAQUS software as given in the Figure 9. The model is based on 3D finite element modelling. Since the applied load represents the actual working loads of the structure and are relatively low compared to the ultimate limit load state of the structure, materials were modelled as elastic and isotropic for all structural elements, matching the concrete class C25/30 and steel grade B500. Special efforts were made to make this model geometrically identical with the real bridge while using reasonable amount of time for modelling.

Numerical model consists of 5 main girders with "T" cross sections and the roadway slab above it. Connections between individual structural elements were defined as stiff and idealized supports were used to represent a simple span beam boundary conditions, Figure 9.

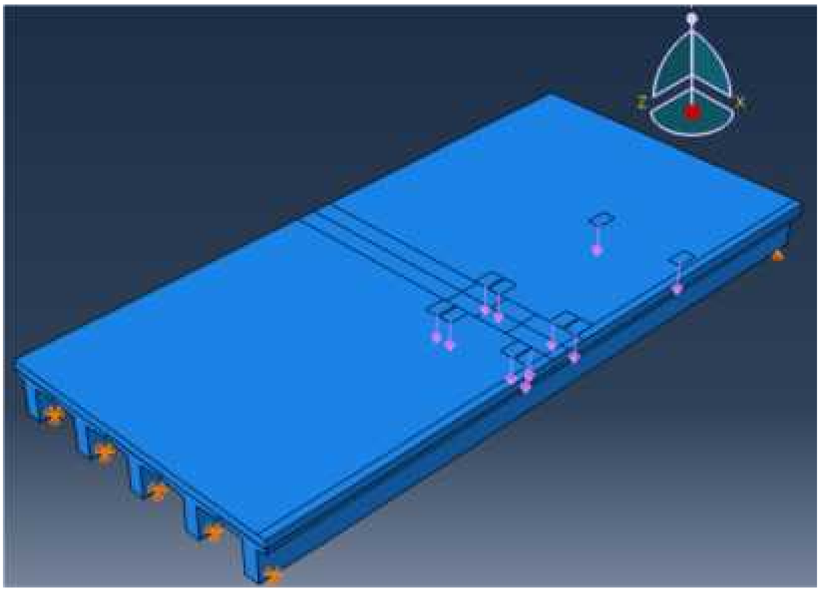

Fig. 9. *3D numerical model of the bridge in Abaqus*

Presented numerical model in Abaqus yielded following stresses for the middle span cross section of the main girder GN2, Figure 10:

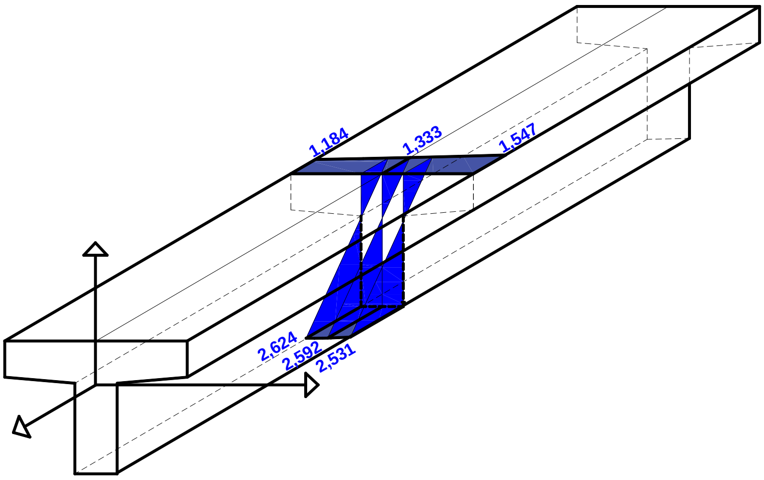

Fig. 10. *Stresses in the mid-span cross section of the girder GN2 [MPa]*

3.3. Stresses Obtained with Simplified Quasi 3D Finite Element Model

In order to investigate the possibility to create a simplified numerical model, with easily readable results, and full use of automatic design procedures available with modern software, a quasi-3D numerical model was defined within the Tower software, Figure 11 and 12. The model consists of 5 longitudinal girders that are represented by line elements with appropriate cross section properties that correspond to composite action of the prefabricated girder and adequate portion of the roadway slab. For longitudinal direction, these girders represent full cross section of the bridge. Interaction between the girders and lateral load distribution were simulated with a slab modelled with 2D finite elements. The thickness of the slab is defined from the average thickness of the joint thicknesses of the roadway slab and the upper flange of the longitudinal girders. This slab is, numerically, positioned in the longitudinal girder's centroid thereby minimizing slab's contribution to the bending in longitudinal direction. This slab is basically used only to simplify the application of the loads that act on the upper surface of the bridge and approximate the lateral load distribution between the longitudinal girders while having a minimal effect on the longitudinal bending. As a result, a numerical model is obtained that is very simple to create, cross sectional forces can be displayed in a manner that most engineers in the practice find clear and easy to utilize and it still keeps all the benefits that modern structural design software offer.

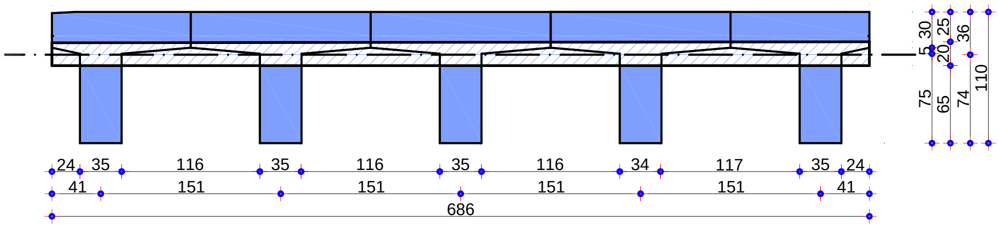

Fig. 11. *Cross section of quasi 3D numerical model*

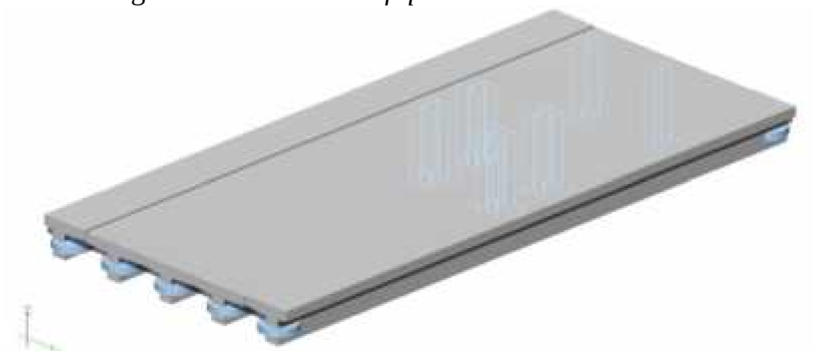

Fig. 12. *Full view of the quasi 3D numerical model of the bridge*

Quasi 3D numerical model, as described here, yields following stress values for the mid-section of the longitudinal girder GN2 are given in the Figure 13.

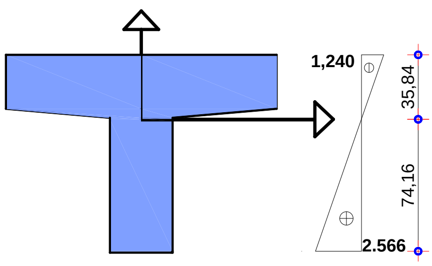

Fig. 13. *Stress diagram for the mid-section of the girder GN2, quasi 3D numerical model, Tower [MPa]*

3.4. Stresses Obtained from the Testing of the Bridge with the Trial Loads

For the purpose of the technical approval of the bridge, prior to its full use, Serbian codes impose demand that a testing of the road bridges with spans over 12 m need to be performed. Keeping this in mind, two bridges with structural properties as described in this paper were tested by trial loads, as shown in the Figure 3. The detailed reports regarding these tests, prepared by the authors of this paper, are given in the reference [7]. Among other results, for the purpose of this paper, the test results provided following stresses within the mid-section of the girder GN2, Figure 14.

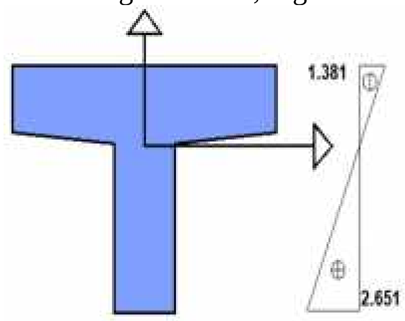

Fig. 14. *Stress diagram for the middle cross section of the GN2 [MPa] according to tests with trial loads*

4. Comparative Analysis and Conclusions

When compared to testing of the real structures, shown numerical results gave following results:

Comparative results of stresses GN2 Table 2

Method	Courbon	Abaqus	Tower	Experiment
Upper flange (MPa)	1.462	1.240	1.333	1.381
Lower flange (MPa)	3.026	2.566	2.592	2.651
Difference, compared to the experiment (%)	+5.9%, upper	-10.2%, upper	-3.5%, upper	-
	+14.1% lower	-3.2% lower	-2.2% lower	-

The results presented in this paper show that complex 3D structural analysis, when used for the purpose of analysing working behaviour of structures under relatively low stresses compared to ultimate limit states, do not always justify a huge amount of work that needs to be invested into numerical modelling. On the other hand creating models that have geometrically realistic appearance can lead to wrong conclusions if the modelling includes oversimplifications in boundary conditions and material modelling. When structural design needs to be easy to analyse, conventional models have certain advantage but they still require significant amount of hand calculation in order to prepare input values for the analysis. As shown here, quasi 3D models are easy to create, insure full advantages that modern software offer, and have satisfactory level of compliance between the results they offer and the actual tests results.

Acknowledgement

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