Investigation of the behavior of the cable-stayed bridge under test load

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A field load testing is an effective method for understanding the behavior and fundamental characteristics of cable-stayed bridges. This paper presents results of the behavior of the reconstructed cable-stayed bridge over river Danube in Novi Sad under test load. The bridge was built between 1976 and 1981 and in 1999 was heavily damaged by two Tomahawk missiles. In 2003 process of the reconstruction began and at the end of 2005 Faculty of Civil Engineering, University of Belgrade was invited to test reconstructed bridge structure. The results of static load testing presented herein include displacements, rotations and strains of the 351 m long main span. Vibrations of the bridge were obtained by impact load produced by heavy truck passing the bridge and the time history response of vertical accelerations was recorded. The frequency content of the signals was determined using Fourier transformation and five flexural natural periods were extracted. The geological structure and engineering properties of the soil are also given. A finite element model of the bridge was made and a good agreement is achieved between the experimental and analytical results. These results have shown that the bridge is in the elastic state under the code-specified serviceability load, which indicates that the bridge has adequate load-carrying capacity and can be put safely into service.

Keywords: cable-stayed bridge, experimental analysis, numerical analysis

1. Introduction

There are many papers related to the investigation of behavior of cable stayed bridges (Gattulli et al., 2002; Weon et al., 2007). Although sophisticated finite element methods are currently available for this purpose, successful analysis is strongly dependent on the experimental verification of the results (Cunha et al., 2001; Santos and Min, 2007; Wei-Xin et al., 2007). A field load test is an essential way to understand real behavior of such kind of bridges. Test results of long span cable stayed bridges are very important and valuable data, which could be used by the other authors to check and improve their analytical methods for calculation the behavior of cable stayed bridges.

Figure 1. Damaged bridge structure.

In this paper static and dynamic behavior of the reconstructed Sloboda Bridge over the Danube River in Novi Sad, under test load is presented. The tests were performed by the University of Belgrade to experimentally identify the most relevant bridge parameters and to correlate them with the corresponding parameters provided by the numerical finite element model.

The Sloboda Bridge was designed by Professor Nikola Hajdin and the Kirilo Savić Institute from Belgrade. The main contractor was Mostogradnja (Belgrade). The bridge was built between 1975 and 1981. On April 4th, 1999 two Tomahawk missiles launched by NATO forces heavily damaged the bridge structure.

Initial assessments of the reusability of the demolished bridge were made in 1999 and 2000. In 2003, the process of reconstruction began, and it was completed in 2005. The details of how the bridge was dismantled and reconstructed have been published previously (Stipanić, editor, 2004; Hajdin et al., 2002). In September, 2005, The Faculty of Civil Engineering, University of Belgrade, was invited to test the reconstructed bridge structures using trial static and dynamic load. The main aim of the bridge testing was to certify the safety of the structure prior to putting it into operation (Ćorić, 2005).

2. Description of the bridge

Detailed descriptions of the bridge have been published previously (Hajdin, 1979; Hajdin, 1983); only the most important data will be given here. The bridge is composed of a main cable bridge and approach structures on the left and right Danube banks. The main cable-stayed steel structure has a 60 + 60 + 351 + 60 + 60 m span arrangement. The pylons (60 m in height) are embedded into the deck structure. The cable stay system consists of six groups of stay cables, each composed of four stays positioned in the central vertical plane. At the time of the construction, the 351 m main span was the largest span in the world for a cable stayed bridge with stays in a single vertical plane.

Figure 2. Reconstructed Sloboda Bridge.

The deck structure consists of a three cell closed box girder with side cantilevers (Figure 3). The depth of the girder structure is approximately 3.8 m. The total width of the orthotropic deck is 27.68 m. Two internal vertical webs enable a transfer of internal forces from the deck structure to the cables and then to the pylons.
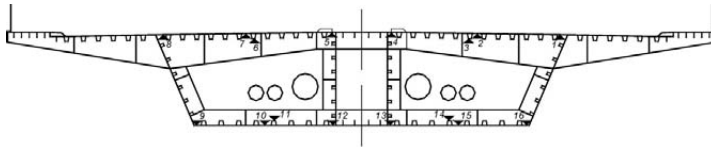

Figure 3. Deck structure of the bridge.

The approach structures on the left Danube bank consist of two continuous pre-stressed girders with a total length of 301 m and four composite beams of 4×60 m. On the right river bank, there are three composite approach beams with spans of 3×60 m.

3. Soil characteristic and foundation system

The properties of the soil in the area around the bridge foundation were investigated before and during construction for different purposes by the following groups: Geosonda of Belgrade, Industroproject of Zagreb, Civil Engineering Institute of Croatia, Zagreb, The Water Resources Development Institute Jaroslav Černi from Belgrade, Hidrozavod DTD of Novi Sad and The Institute for Testing Materials of Serbia (IMS).

These investigations included stereoscopic analysis of aerial photographs, interpretation of satellite images, a quantitative geomorphologic study, detailed engineering geological mapping, exploratory drilling and trenching, cone

penetration tests, hydrogeological and hydrometeorological observations, installation and monitoring of piezometers and inclinometers, pumping tests for soil permeability *in-situ* and laboratory soil mechanical tests (Saković, 1993).

The bridge is located on the Danube bank slope. The heterogeneous lithology of the surface has been formed by both old and recent endogenic and exogenic geological processes, as well as by human impacts. The following effects of the exogenic processes are prevailing: surface rock weathering, slope erosion, and both eolian and fluvial processes.

There are three principal landforms, spacious plateau, varied slope and alluvion. Windblown loess on the plateau has greatly subdued the relief-to-height difference from 139 to 143 m.

The bridge area was built of Pliocene and Quaternary deposits; the oldest materials (to the exploration depth) are the Pliocene shallow lake sediments represented by grey and grey-yellow clayey marl and silt with lenses and intercalations of sand and clay. Smaller proportions of black soft clay, reddish marl and lignite seams also exist. All of these deposits are stratified and inclined to the Danube at angles from 2° to 5°. Upper portions of the deposits are altered by weathering and deteriorated over time. In addition, they are enriched with iron and manganese oxides and calcium carbonate.

The sediments composing the terrain to the explored depth are varied in composition and age. In addition, they were deposited under different sedimentation conditions (deep water, shallow water, swamp, dry land) and subsequently were exposed to intensive engineering geological processes; the result was high heterogeneity and anisotropy of the engineering-geological and hydrogeological properties in the given area.

Rocks are classified on the basis of engineering geological mapping and core examination into the following seven geotechnical units, (Figure 4):

P l i o c e n e
- Fresh lake deposits (1)
    - Marl and clay (1/1)
    - Sand and silt (1/2)
- Altered, deteriorated deposits (2)
    - Weathered marl (2/1)
    - Soft clay (2/2)
- Clayey sand (3)

Q u a t e r n a r y
- "Srem" sediments (4)
- Loess deposits (5)
- Recent slope erosion products (6)
- Alluvial deposit (7).

1. Fresh Pliocene marl and clay   4. "Srem" sediments   7. Alluvium

2. Weathered Pliocene marl and clay   5. Loess deposits   - - - - Failure plane

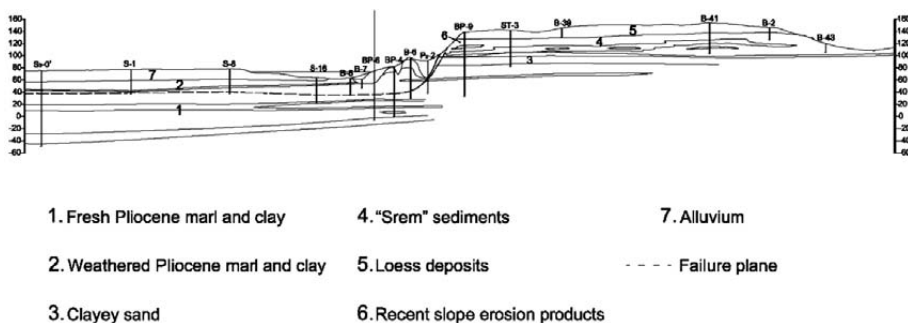3. Clayey sand   6. Recent slope erosion products

Figure 4. Engineering geological cross section.

The numbers in parentheses designate respective geotechnical units listed in the table of the mechanical properties of soil (Table 1).

Classification parameters, as well as values of unconfined compressive strength and shear strength by drain tests, were derived on the basis of statistical analysis of results of laboratory testing. The performed tri-axial compression and odometer tests were inadequate for proper statistical processing; therefore additional test data were analyzed.

The highly heterogeneous hydraulic properties of sedimentary deposits and the activity of the currently evolving geologic processes have given rise to a varied hydraulic conductivity ratio of sediments, even within the same lithologic unit. Loess is a soil of high vertical hydraulic conductivity and appreciable porosity, which enables atmospheric precipitation to infiltrate without being retained. Srem series soils have varied hydrologic properties, from those of an aquifer, to an aquitard, to a local groundwater barrier. These soils form two aquifers at depth intervals 21.3–25.4 m and 30.8–35.5 m. The groundwater drainage in morasses and at springs indicates water movement from the loess plateau inland to the Danube bank slope in the bridge area. A sand-gravel series was deposited in shallow water after the Pliocene hiatus, and it act as sub-artesian aquifers. Three major sand aquifers have been found in the depth intervals 45.8–47.0 m, 57.2–59.2 m and 75.9–85.0 m. An excess water pressure was measured by piezometers in each aquifer. The aquifers are separated by thick impermeable marl and clay marl.

Geodynamic processes that have evolved in the bridge area are surface weathering (area of weathered clayey marl and marl), slidings, planar and linear erosion, wind erosion (loess deposition), fluvial erosion (the Danube right bank, consequent river channel shift to the south and deposition of sediment load transported by traction), and delluvial erosion.

A wider bridge area on the right side of the valley is designated as unstable, associated with a slide almost 40 m deep and of complex descent and differential block movement. The sliding process of variable intensity is evolving

Table 1. Mechanical properties of the soil for different geotechnical units.

Geotechnical complex		Fresh sediments (1)		Altered sediments (2)		Clayey sand (3)	"Srem" sediments (4)	Loess sediments (5)	Deluvial silty clay (6)
		Marl and clay (1/1)	Sand and silt (1/2)	Weathered marl (2/1)	Soft clay (2/2)				
Unit weight (kN/m³)	γ_s	25.7	26.3	25.54	24.90	25.83	24.99	25.56	25.63
	γ	20.3	20.05	20.13	19.36	19.75	20.63	19.78	20.62
	γ_d	16.8	15.9	16.72	15.47	16.32	17.40	16.18	17.15
	γ_z	20.4	20.03	20.35	19.48	20.08	20.77	19.90	20.67
Percent particle size (%)	Clay	19	5	19	32	7	21	13	17
	Silt	70	44	67	62	29	57	81	61
	Sand	11	51	14	6	62	22	6	21
	Gravel	-	-	-	-	2	-	-	1
Water content (%)		20.60	24.97	20.75	25.46	20.86	19.22	18.37	20.24
Porosity (%)		34.50	39.82	34.37	37.09	36.14	30.37	36.66	33.11
Atterberg limits	LL	51.90	36.52	57.88	75.66	46.72	48.79	39.66	46.0
	PL	23.04	21.40	24.56	29.21	22.57	20.58	20.85	21.28
	PI	28.86	15.12	33.32	46.45	24.15	28.21	18.81	25.09
Unconfined compressive strength, q_u (kPa)	$q_{u\ max}$	220.50	158.50	253.00	364.70	64.00	341.00	-	171.30
	$q_{u\ 15\%\ def}$	120.00	90.00	140.30	282.83	-	235.80	-	135.00
Shear strength parameters	φ' (°)	12	24	9.5	6	21	14.5	16.5	12.5
	c' (kPa)	20	0	0	0	0	20	4	0
	φ (°)	22	-	17	8.5	0	-	-	-
	c (kPa)	73	-	14	54	-	-	-	-
Modulus of compressibility (MPa)	$M_{s\ 200\text{-}400}$	20.62	-	20.12	25.64	-	-	-	-
Coefficient of permeability (cm/s)	k_{fV}	5.36×10^{-7}	8.18×10^{-4}	-	-	3.4×10^{-5}	-	-	-
	k_{fH}	1×10^{-6}	8.86×10^{-5}	-	-	1.7×10^{-3}	-	-	-
	$k_{f\ USBR}$	2×10^{-7}	2×10^{-5}	2×10^{-7}	$<10^{-7}$	8.5×10^{-5}	$<10^{-7}$	-	3×10^{-7}
	Field test	-	-	-	-	6×10^{-5}	6×10^{-5}	-	4.6×10^{-5}
Swelling pressure (kPa)		-	-	75–350	350	-	-	-	-
Coefficient of earth pressure at rest, K_o		-	-	0.48–0.64	0.8	-	-	-	-

across a zone of 400–600 m. In the immediate bridge area, a block about 200 m wide is affected by sliding. The sliding body has a front at a steep, sub-vertical scarp, which is the boundary between the unstable and stable soils of the broad loess plateau in the bedrock. The slide bottom enters deep under the Danube bed and ascends at a low angle some 100–150 m from the right riverbank. Soil slides along the geological contact between fresh and weathered sediments. The shear surface has a slope angle of about 3° and it is located within the layer of saturated soft clay of high plasticity with low-strength parameters.

In the area affected by sliding, piers S17 and S18 through S21 are founded on piles. Soil consolidation involved a complex drainage system of two wells that are 6.5 m in diameter and 21–24 m deep, eight horizontal drains that are 0.47 m in diameter and 205 m in total length, and a tunnel outlet that is 1.5 m in diameter and 139 m long (Jelisavac et al., 2001).

Mechanical properties of the soil for different geotechnical units are presented in Table 1.

The foundation system is composed of reinforced bored piles with diameters $\varnothing = 1200$ mm $\varnothing = 1500$ mm and connected using large RC pile caps. For the foundations of the main bridge (between piers 14 and 19, Figure 2) the following arrangement was used: $10 \varnothing 1200$ (pier 14), $8 \varnothing 1200$ (pier 15), $32 \varnothing 1500$ (pier 16 – pylon pier), $24 \varnothing 1500$ (pier 17 – pylon pier on the right river bank), $8 \varnothing 1200$ (pier 18), $10 \varnothing 1200$ (pier 19). The length of piles varies between 20 m on the left Danube bank and 35 m on the right bank.

Bellow the piers and pile caps, the piles with large diameters were modeled with vertical beams loaded at the top and supported by deformable medium characterized by horizontal and vertical moduli of subgrade reaction. The soil under the bottom of the piles is simulated by vertical springs and the soil around the pile shafts is simulated by series of horizontal springs. The moduli of subgrade reaction were estimated using solution proposed by Vesić (1961). Modulus of elasticity of soil were determined from the results of unconfined sompression tests of soil. On the basis of that, for modeling of substructure soil, for static analysis was assumed $E = 80$ MPa. With regard to the type of structure, for dynamic analyses was used $E = 120$ MPa.

4. Numerical analysis

During the investigation studies, which were performed during 2000 by the Kirilo Savić Institute, numerical simulations were performed to assess any damage to the foundation structure. Analyses of the nonlinear response of the piles and the foundation structures of the heavily damaged piers (S14 and S15 on the left river bank) were performed using geotechnical data (presented in this paper) and assumed collapse mechanism (Hajdin et al., 2001).

The numerical analysis of the rebuilt bridge under the test load was performed by the authors of this paper. It should be mentioned that the results of

Figure 5. Finite element mesh of the bridge superstructure.

this analysis are used only for the control of the behavior of the reconstructed bridge under load test. A spine finite element model with beam and truss elements was made, Figure 5. The results of soil mechanic investigations presented in Section 3 were also applied in numerical model. By several numerical analyses, with large range of moduli of subgrade reaction, it was found that the soil and substructure stiffness have small influence on measured data of the structure. This is expected behavior due to the flexible superstructure with non rigid connection of bridge girder with massive concrete piers and relatively small displacements of substructure, too.

The bridge girder was discretized with 3 m long beam elements, and a pylon a mesh with 6 m beam elements was used. The geometric data, including cross sectional properties, were taken from the main design performed during bridge reconstruction. The cables were modeled with truss elements, taking into account sag effects by introducing effective cable modulus. The effective cable stiffness is stress dependent; therefore, a two-step analysis was performed to evaluate the response of the main bridge due to test loading. In the first step, an estimated existing superimposed dead load was applied using a model with cable stiffness based on forces achieved during construction. An iterative procedure was used to ensure that the effective cable modulus corresponded to the cable forces at the start and the end of the loading step. In a second step, the load arrangements that correspond to the test phases were applied using the effective cable stiffness based on the cable forces from the previous step. Results of numerical analysis are given in Section 5.

5. Experimental analysis

At the end of reconstruction, before opening for traffic, the bridge was subjected to a load test. The bridge was tested using a trial load in accordance with code JUS U.M1.046. Dynamic characteristics of the structure were investigated together with global and local deformations of the bridge. Vertical deflections, horizontal displacements, rotations and strains were measured on the main bridge and two approach girders.

Vertical displacements were measured in 23 cross sections of the bridge by surveying-leveling. Displacement of the top of the pylon was measured using GPS technology, with a TRIMBLE 4600LS receivers placed on the top of both pylons, as presented in Figure 6. The sensors received signals every 15 sec-

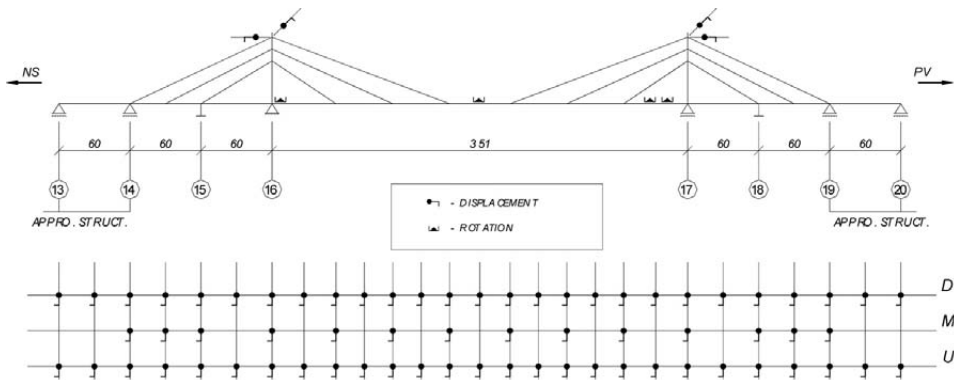**Figure 6.** Displacement was measured using GPS technology.

Figure 7. Position of measurement points for static tests.

onds. The world coordinate system (WGS84) and appropriate software were used for vector analysis of displacement. Maximum horizontal displacement of the top of the pylon obtained by test was 55 cm. Calculated value is 58 cm.

Position of measurements points for global deformation is presented in Figure 7.

5.1. Static load test

The static load used was 20 heavy trucks, approximately 42 tones each, as presented in Figure 8. Intensity and positions of trucks for static load were calculated to produce 0.50–0.85% of maximum characteristic stresses.

Figure 8. Trial static load.
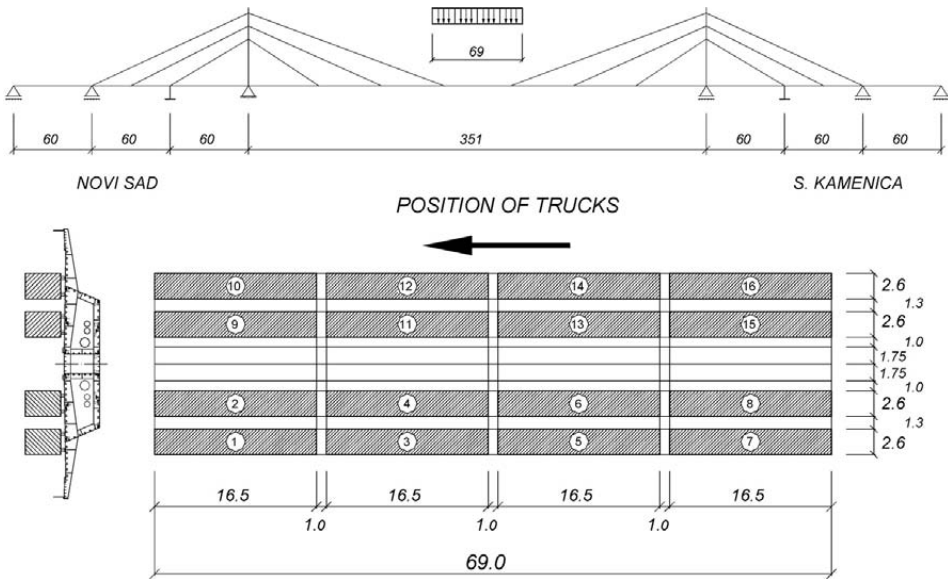
Figure 9. Load phase 1.

Static load was performed by 7 different phases.

In this paper it is presented load phase 1 which produces maximum stress in the middle of the bridge (Figure 9).

Maximum vertical displacements of main girder were obtained in the middle of the bridge and it was $v = 48.2$ cm. Corresponding theoretical (calculated) value is 54.5 cm (Figure 10).

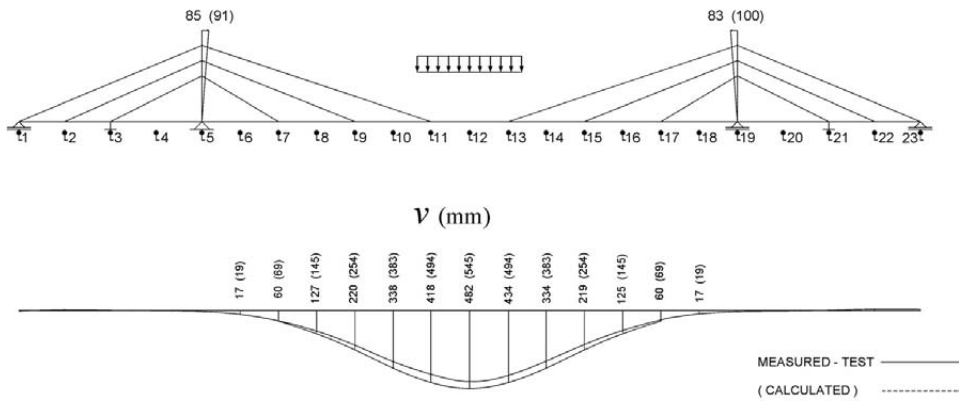Distribution of normal stresses in the middle of the bridge is shown in Figure 11.

Figure 10. Vertical displacements (phase 1).
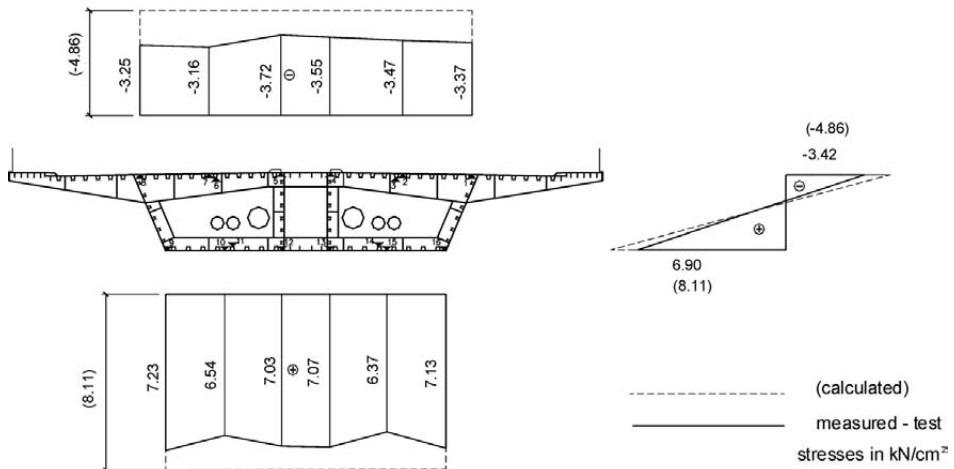
Figure 11. Normal stresses in the middle of the main span.

Non-symmetric load in the phase 2, when the trucks are only in one lane, produced maximum torsion of the bridge cross-section. Measured value in the middle of the bridge was $\varphi = 1.04\%$. Theoretical rotation is $\varphi = 1.10\%$ (Figure 12).

5.2. Dynamic load test

Dynamic load testing was performed to find the natural frequency of the main bridge structures. The dynamic load used was a heavy truck (42 tones) at a speed of 40 km/h, which produced an impact load by crossing a barrier (10cm height) that was placed over the middle of the span. Measurement data system acquisition, MGCplus, Hottinger HMB and a vibration-meter (MTN/1100)
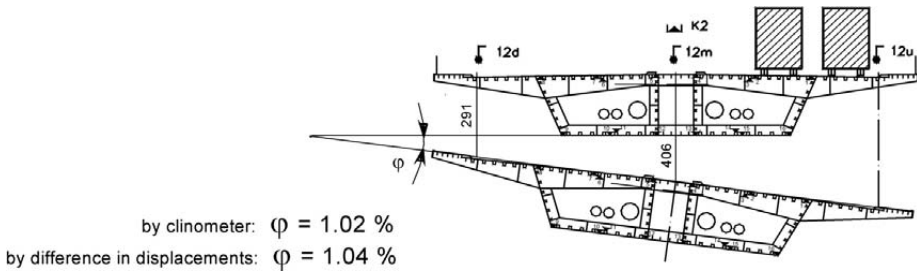
by clinometer: $\varphi_1 = 1.02\ \%$
by difference in displacements: $\varphi = 1.04\ \%$

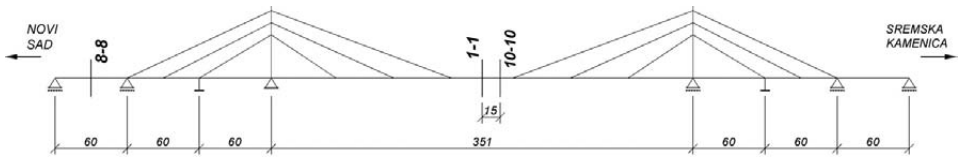**Figure 12.** Torsion (rotation) of the bridge (phase 2)

Figure 13. The measurement stations for dynamic response testing.

were used for dynamic testing. To record vertical acceleration, accelerometers were placed at the top of the deck plate in the middle of the cross section at the following two measurement stations along the span: in the middle of main span (section 1-1) and 15 m away from section 1-1, at the position of the cross frame (section 10-10). These sections are shown in Figure 13.

Dynamic numerical analysis was performed on the system with dead weight only (weight of the structure plus superimposed dead weight). Three characteristic flexural modes are presented in Figure 14.

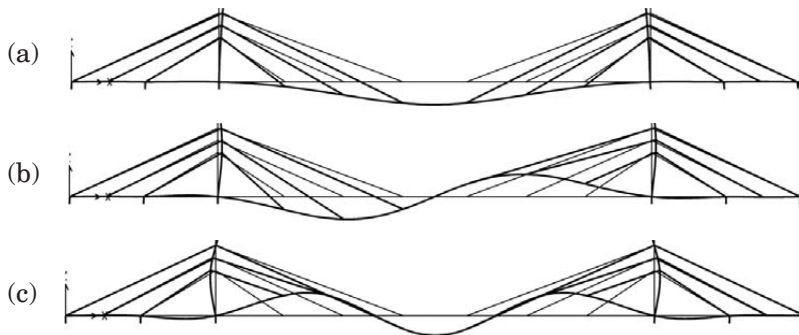Vibrations of the bridge recorded during testing are presented in Figure 15.

Figure 14. (a) First mode ($f_1 = 0.40$ Hz) (b) Second mode ($f_2 = 0.64$ Hz), (c) Fourth mode ($f_4 = 1.00$ Hz).
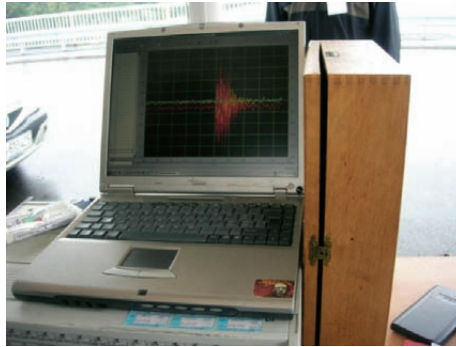
Figure 15. Recording the vibrations during testing.

In Figure 16 the time history of vertical acceleration at two measurement stations are shown.

A Fast Fourier Transformation with $N = 2^{10}$ points spaced at intervals of $\Delta t = 0.02$ s was used to transfer the acceleration signals into the frequency do-
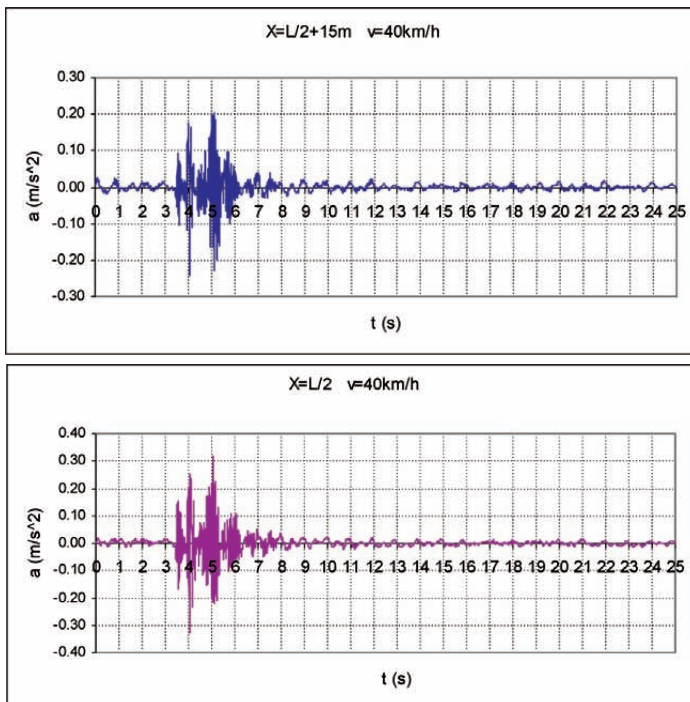

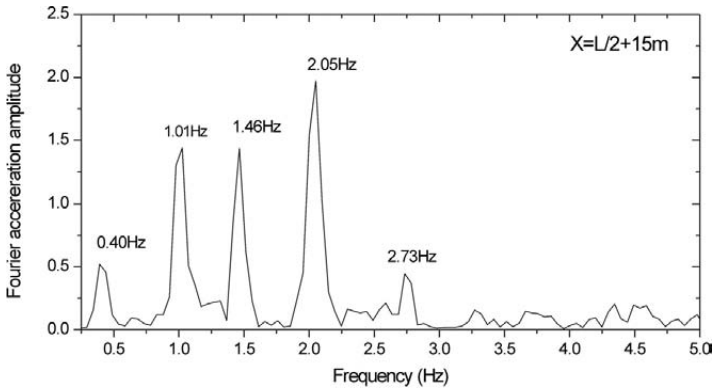**Figure 16.** Time histories of acceleration at two locations.

Figure 17. Amplitude acceleration spectrum – section $X = L / 2 + 15$ m.

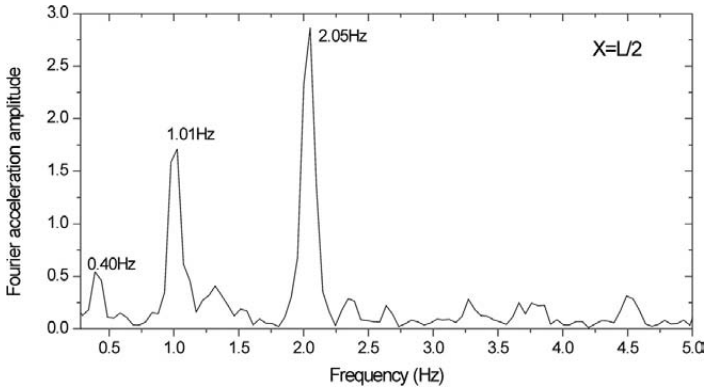**Figure 18.** Amplitude acceleration spectrum – section $X = L / 2$.

main with a step size of $\Delta f = 0.04883$ Hz. The amplitude acceleration spectra for the two measuring points are given in Figures 17 and 18. From the obtained spectra, five natural frequencies were extracted.

Measured and calculated values are presented and compared in Table 2. Good agreement between measured and calculated natural periods was achiev-

Table 2. Measured and calculated natural frequencies of vertical flexural and torsional modes. # – Not extracted from recorded data, x – Not measured (torsional modes).

Mode Number	1	2	3	4	5	6	7	8
Calculated (Hz)	0.40	0.64	0.98	1.00	1.47	1.96	2.08	2.70
Measured (Hz)	0.40	#	x	1.01	1.46	x	2.05	2.73

ed. Note that the second mode (anti-symmetric) was not identified. This result may be due to the fact that at the position of the accelerometer ($x = L / 2 + 15$ m) the theoretical modal amplitudes are very small, because the position was close to the mid-span. The torsional frequencies were not measured. The torsional response can be inferred only from the static tests.

6. Conclusions

On the basis of results obtained by trial load testing and results obtained by numerical analysis, it can be concluded:

– Behavior of the bridge structure subjected to trial load is completely elastic. After the load (trucks) moved away, the deformations returned elastically, without any plastic deformations.

– The performed numerical tests show that the variation of subgrade moduli has no significant influence on measured static and dynamic structural response. Such behavior of the bridge is caused by applied flexible steel superstructure with non rigid bearings system between bridge girder and massive concrete piers with pile caps on bored piles.

– A good agreement is achieved between the experimental and analytical results.

– The values of all global deformations (displacements and rotations) and the values of all stresses obtained by test are smaller than the values obtained by numerical analysis.

– Carrying capacity of the bridge structure obtained by test load is higher than it was predicted in structural design.

Finally it can be concluded that field load test of the cable stayed bridge is a powerful method for better understanding the behavior of the bridge and for checking the main assumptions used in the numerical approach.

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SAŽETAK

Ispitivanje ponašanja ovješenih mostova pod probnim opterećenjem

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Ispitivanje ovješenih mostova opterećenih probnim opterećenjem je efikasan način za sagledavanje ponašanja i određivanje glavnih karakteristika takvih mostova. Ovaj rad prikazuje ponašanje rekonstruiranog ovješenog mosta preko Dunava u Novom Sadu. Most je izgrađen između 1976. i 1981. godine, a 1999. godine bio je teško oštećen kada su ga pogodile dvije Tomahawk rakete. Godine 2003. počela je rekonstrukcija mosta, a krajem 2005. godine Građevinski fakultet Sveučilišta u Beogradu bio je pozvan ispitati rekonstruiranu konstrukciju mosta. U ovom radu prikazani su rezultati koji se odnose na pomake, rotacije i deformacije glavnog nosača konstrukcije duljine 351 m. Vibracije mosta izazvane su udarnim opterećenjem uslijed prolaska teškog kamiona preko mosta, pri čemu su određena vertikalna ubrzanja glavnog nosača. Potrebne funkcije u frekventnoj domeni određene su primjenom Fourier-ove transformacije i na taj način je izdvojeno prvih pet perioda osciliranja. U radu su prikazane geološke karakteristike terena. Formiran je numerički model mosta na bazi metode konačnih elemenata i dobiveno je dobro slaganje eksperimentalnih i numeričkih rezultata. Ovi rezultati pokazuju da se most ponaša elastično pod utjecajem probnog opterećenja, što ukazuje da konstrukcija ima odgovarajuću nosivost i da se sigurno može pustiti u promet.

Ključne riječi: ovješeni most, eksperimentalna analiza, numerička analiza

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