Post - repair dynamic investigation of the Gazela pedestrian bridge in Podgorica, Yugoslavia

Prof. Ljubomir Vlajić[1], PhD; Lect. N. Djuranović[2], PhD and Ass. Duško Lučić[2], MSc
[1] University of Novi Sad, [2] University of Montenegro

ABSTRACT

This paper describes dynamic testing of a three span (13m + 78m + 13m) pedestrian bridge located in the city center of Podgorica, Montenegro. The structural examination and testing was conducted after the repair and strengthening of the structure. The bridge crosses 60m deep (low water) Moraca river canyon. Dynamic testing involved establishment of dynamic movement amplitudes and frequencies, damping coefficient and real values of dynamic increase factor. Measurements were taken for three perpendicular directions. The bridge structure was dynamically excited in several different ways and response was recorded in each case.

One of the main reason for undertaking the strengthening work was an attempt to change the frequency of natural vibration. It is established that remedial measures and bridge strengthening did not change the values of natural vibration of the structure. The measured vibration frequencies stayed still out of the range allowed by the current Yugoslavian regulations for pedestrian bridges.

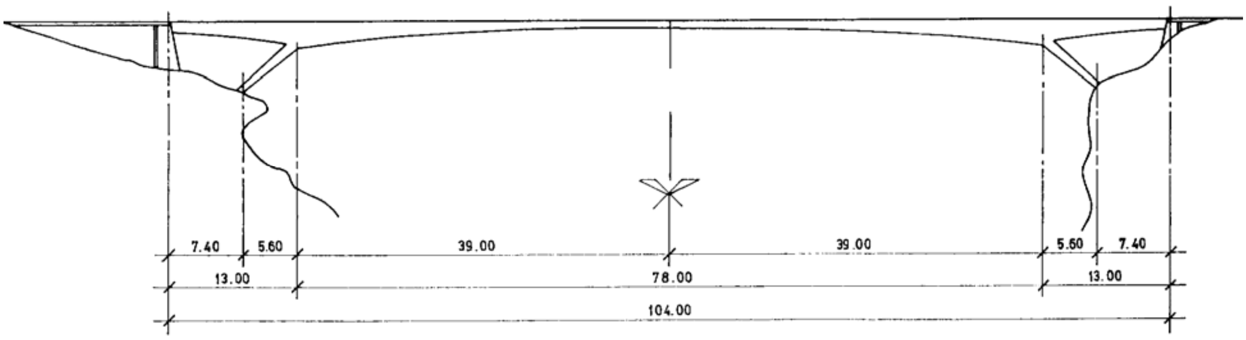
Keywords
Post - repair, testing, dynamic, pedestrian bridge, steel

INTRODUCTION

The primary role of the tested bridge is to serve for pedestrian traffic. However, it also carries main waste water drains from one to the other bank of the Moraca river.

The bridge was originally constructed and proof tested in back 1970, when it was proved to be of satisfactory state and in usable condition (Institute or material testing SRS, (1970). In the meantime the structure considerably deteriorated.

Steel main girder was of propped shore type with box shaped cross-section. It has three spans, 2 x 13.0 m + 78.0 m. It had oblique, structural, props connected to concrete fundaments, with assumption of the full fixity at the support. Main girder end supports were designed and built as "free to move", but also were enabled to take significant "negative" downwards reaction.



Picture 1: Disposition of the bridge

Main girder cross-section consisted of a trapezium - shaped box with superstructure top slab of constant width. Since both sides of the main box girder had the same inclination, and since they were of parabolic shape, the substructure slab was of a variable width.

The height of the main box girder at the middle and at the ends of the bridge is 1.400 mm, whilst at the place of leg (prop) supports it is 2.800 mm.

Steel made top slab of the box was 12 mm thick. It was additionally strengthened with longitudinal ribs. By doing this, the bridge upper deck is formed as an orthogonaly anisotropic slab.

Inclined sides of the main box girder, were made of tin plates, and also strengthened with longitudinal stiffeners, placed at the thirds of the plate heights. At the place of bridge props, the main girder lower deck width was 1600 mm.

In order to provide structural stability, the oblique legs i.e. props, were pushed aside, when looking towards the

RC fundament. At the foundation level their open spacing is 3,00m.

The main reason for conducting repair and strengthening work was to reduce stresses in oblique bridge legs. These stresses were very close to allowable limit values, for at least one of the possible load combinations. Another aim was to reduce, to a certain degree at least, the existing vibration problems which could cause the feeling of discomfort to the bridge users. The vibrations were particularly evident when a small number of pedestrians (one or two) use the bridge.

REPAIR AND STRENGTHENING MEASURES

Problems of pedestrian bridge vibrations are usually avoided by designing the structure in a way that its natural frequency does not come to the range of 1,6 to 2,4 Hz. According to Swiss code of practice SIA 160, the entire range of 3,5 Hz to 4.5 Hz should also be avoided. Yugoslavian code regulations (Official Bulletin of SFRJ 1/91, 1991) require that, in order to fulfill serviceability limit state conditions, "the frequency of unloaded pedestrian bridge must not be in the range of 0,8 Hz to 5,5 Hz". The code of practice current at that time when this bridge was designed and built, did not contain this provision.

Today, at least three ways of eliminating or reducing pedestrian bridge vibration problems are available.

When trying to reduce vibration problems of pedestrian bridges, repair work is usually conducted in one of three ways:

- by providing additional strengthening (stiffening) of the bridge,
- by increasing the amount of damping existent in the bridge structure, and
- by providing (connecting to the bridge structure) vibration absorbers.

It is recommended that additional strengthening should be undertaken if the bridge stiffness is not over 8 KN/mm (Number of authors, 1995).

Usually the most economical of all possible measures is to increase the damping level in the bridge, so the structure can absorb more energy without significantly exciting the bridge. This can be achieved through the whole series of actions, starting from resurfacing the existing pavements with soft asphalt coats, changing or artificially modifying the existing supports and bearings in order to secure higher damping.

Vibration absorber method is based on adding to the bridge structure a new, artificial, oscillating system. It is usually one degree of freedom spring & mass system whose basic frequency (the first mode) should be the same as that of the bridge itself. By conducting necessary analytical work it is possible to prove that the mass of such a system usually ranges from 0,05% to 1% of the bridge mass. The problem usually associated with this remedy solution is the provision of the necessary space for the added vibration system. That space should be big enough to accommodate movement of the additional oscillating system, without affecting usability and esthetic aspects of the bridge.

Repair and strengthening work conducted on this bridge mainly consisted of strengthening bridge props (legs) by steel plates. In addition, a concrete slab was cast inside the main box girder, to 5.20m left and right from the point were props were connected to the main girder. Also asphalt resurfacing of the top slab was undertaken.

BRIDGE TESTING

The main aims and purpose of this testing was to:

- establish real static and dynamic behavior of bridge, under different service conditions;
- determine stresses and deformations in the structure, as well as parameters which describe dynamic behavior of the system;
- establish and verify the structural model and assumptions used in the structural analysis and design of the bridge;
- make reliable and explicit conclusions on bridge structural safety and reliability, it's serviceability limit state and overall usability;
- make a proposal for future bridge maintenance and possible further strengthening work.

Static load tests consisted of monitoring overall and local behavior of the bridge, and its parameters (Faculty of Civil Engineering, 1998). This part of the testing program will not be considered in detail on this occasion.

In dynamic tests, according to Yugoslav standards for testing of bridges (JUS U.M1. 046/84, 1984), it is necessary to establish vertical deflections in the middle of chosen spans. The measurements should be taken at several time steps during the application of the load. Also, dynamic characteristics of the loading (for example the speed of load crossing the bridge) should be monitored and recorded.

Other measurements that should be taken in dynamic tests include:

- deflection and strain measurements at other places of anticipated extreme stress and load condition;
- measurement of cross-sectional and longitudinal movements at the middle of chosen spans;
- measurement of other dynamic parameters of the structure.

There are three basic criteria used to establish whether tested bridge satisfies necessary requirements.

The bridge is declared to satisfy dynamic testing criteria if its measured natural period of vibration $\boldsymbol{t_{o,exp}}$ is "close" to its theoretical value $t_{o,des}$ i.e. they should be approximately the same:

$t_{o,exp} = t_{o,des}$

The next condition provided in this code is that the dynamic increase factor $K_{d,exp}$ (defined from the dynamic test results) stays within the limits of value used for the structural analysis and design of bridge, $K_{d,des}$

$K_{d,exp} \approx K_{d,des}$

The most important evaluation criteria from the aspect of dynamic testing for this bridge is that vibrations should not be such to make users have the feeling of discomfort. This is usually the result of the bridge trembling, as well as of certain combination of amplitudes and frequencies which could cause this unpleasant feeling. The only measures of uncomfort are recorded vibration velocities and accelerations and their comparison with previous experience and current requirements.

In principle, the structure would be declared unfit to pass technical quality criteria if a single one of the quoted requests is not fulfilled.

Dynamic testing on this particular bridge, included monitoring of the following parameters:

- displacement amplitudes in vertical, horizontal - transverse, and horizontal - longitudinal direction;
- natural vibration frequencies of the structure. Vibrations induced by providing sudden movement in three, above mentioned orthogonal directions;
- damping coefficient, and
- real values of dynamic increase factor.

All those measurements were taken in the middle and quarter points of the main bridge span. The parameters were monitored simultaneously in three orthogonal directions, so that at each moment true displacement vector could be determined.

Real values of dynamic increase factor were calculated from the measurements taken at the middle of the main span.

Static (dead) proof load was provided by means of four FAP trucks, with individual mass of about 9.0 t (Faculty of Civil Engineering, 1998). The trucks were positioned according to previously made test program, in a way that extreme stress conditions are caused. In essence, trucks were placed to positions required by corresponding influence lines.

Picture 1. Position of the proof load during the structural testing

Maximal value of sagging moment ($M_{max, exp}$ = 1.732,10 kNm) was measured in the middle of the main span. The calculated value obtained for the ultimate design loading of p = 5.0kN/m^2 is $M_{max,des}$=2.880 kNm. Consequently "proof load efficiency coefficient - U" (as described by Yugoslavian standard for testing bridges - JUS-UM1-046) is U = 61,18%. This value was well inside required limits of $0.5 < U < 1.0$.

Dynamic testing was performed for the following load cases:

- Overall dynamic effect. This was produced by a single truck moving across the bridge.
- Blow effect. Impact to the bridge was produced by the same heavy truck going over an obstacle. The obstacle was a wood plank, about 50 mm thick.
- Horizontal longitudinal excitement. Initial dynamic movement was produced by truck moving across the bridge and then breaking at the speed of 5 km/h.
- Vertical excitement. This was produced by one or more pedestrians walking on the bridge.
- Horizontal transverse excitement - induced by sudden release of a tensioned string connected to the middle of the bridge . The tensioned string was used to take the bridge out of balance position in this direction.
- Ambient vibrations. The only excitation to the structure was slight wind, and finally
- Normal usage conditions. This condition was obtained by intensive pedestrian traffic.

Portable vibration measuring instruments - VIBRATION MEASURING UNIT - SMU 31, product of HBM company, Germany, were employed for registering all dynamic vibration effects on the bridge (HBM, date unknown). These devices measure vibration amplitudes, speeds and accelerations. Its measuring device consists of a mounting flange, sensing probe and an indicator unit.

The electrodynamic sensing probe – transducer, with its damped spring - mass system produces an electrical signal which is proportional to vibration velocity of tested structure. In order to obtain movement amplitudes such signal was automatically integrated (in the indicator unit itself), whilst, in order to obtain accelerations of the structure, the signal was differentiated in the same way.

Indicator units also enabled connection of the measuring devices to AD converter, from where digitized signal was sent to a PC for further processing.

The whole testing procedure, probably due to clear and distinguishable structural system, gave high quality results.

TEST RESULTS AND ANALYSIS

Dynamic test results for one of the conducted tests is shown in the following figures.

For all the presented results, measuring devices were positioned in the middle of the bridge, so that they could register vertical, horizontal, transversal and horizontal longitudinal vibrations.

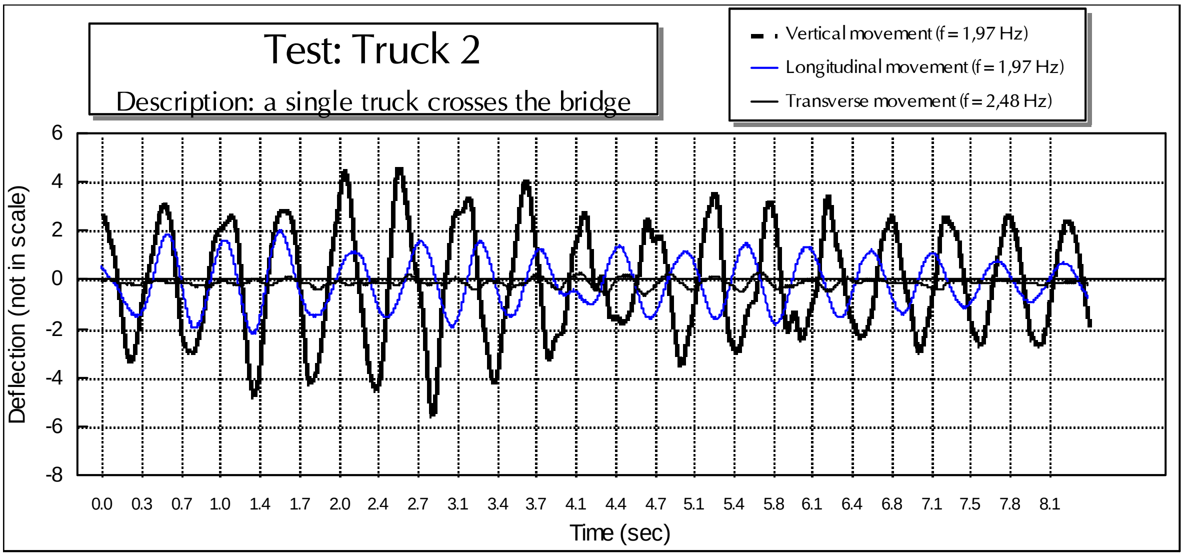


Figure 1 - Displacements - time diagrams.

Maximal value of dynamic increase factor $K_{d,max}$ determined for artificially caused blow effect (one vehicle crossing over 50mm high obstacle) was $K_{d,max}$=1.23. Damping coefficient values could not be easily and reliably determined since the vibrations were damped down very slowly, implying very low logarithmic decrement.

The process of obtaining (determining) natural frequencies of the tested structure was conducted by using "in house" made software, which applies Fourier signal analysis for determining dominant frequencies (Fig. 2).

Experimentally established natural frequency of the structure for the vertical direction was $f_{o,test}$ = 2.0 H_Z, whilst corresponding value calculated for the design purposes was $f_{o,des}$=1.92 H_Z

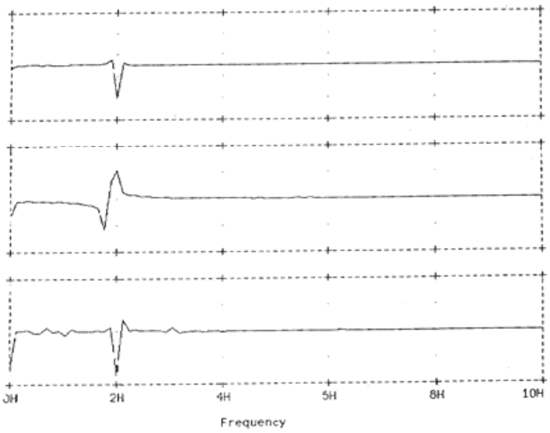


Figure 2 - Typical results of the Fourier signal analysis

However, it is necessary to emphasize that these values refer to frequencies of the first, basic, mode which, in this case was confirmed to be symmetrical (Faculty of Civil Engineering, 1998).

On the basis of the frequency response analysis it was established that:

$$\sqrt{\frac{\max \delta_{rac}}{\max \delta_{mjer}}} \cong \frac{{}_{mjer} f_0}{{}_{rac} f_0}, \text{ i.e. } \sqrt{\frac{\max \delta_{rac}}{\max \delta_{mjer}}} \cong \sqrt{\frac{47.5}{42.7}} = 1.0547$$

It follows that $f_{o,test}$ should be 2,02 H_Z, which proves extreme accuracy of the conducted experimental measurements.

CONCLUSIONS

The pedestrian bridge was tested in accordance with current Yugoslavian testing regulations provided in JUS-UM1-046/1984. Proof load efficiency coefficient U was 61,18%.

Structural analysis model used during the design procedure was fully verified. It quite reliably describes the real structural behavior observed during static and dynamic testing of the bridge.

The undertaken remedy measures reduced stress levels in some sections of the structure, but only for certain number of loading conditions. Stresses in the structure caused initially by original dead load, erection assembly procedures, and also those induced by additional load (concrete inside the steel box and additional tin plates) remain in structure as residual stresses. Overall stresses were slightly reduced, but only in small number of cross-section of the structure. In this respect repair effects were almost negligible.

Repair and strengthening of the structure did not influence the natural frequencies of the structure, either. The frequency was still out of permissible range given in current Yugoslavian regulations for pedestrian bridges. Natural frequency of the structure in the first mode of oscillation (vertical direction) did not change after the repair. So they were still: $f_{o,exp}$= 2.0 H_Z It has to be stressed that at the time when the bridge was originally designed there were no code provisions regarding bridge frequency requirements (MIN, 1969). Current pedestrian bridge regulations do not allow that value to be in the range between 0.8 and 5.5 Hz (Official Bulletin of SFRJ 1/91, 1991).

The structural stability and the ultimate limit state of the bridge were such that the structure could carry all design loads with the sufficient safety factors. Both, local and global strength are secured.

The bridge structure was declared fit for in-service use with remark that the pedestrian crossing the bridge are going to experience unpleasant feeling because of high amplitudes (1.7mm), low values of natural frequencies (2.0 Hz) and small damping factor.

REFERENCES

JUS U.M1. 046/84. (1984). Proof testing of bridges, Official bulletin of SFRJ 60/84, Belgrade.

Number of authors. (1995). Vibration problems in structures - Practical guidelines, Birkhauser Verlag, Bassel.

Faculty of Civil Engineering. (1998). Report on static and dynamic proof testing of steel bridge "Gazela" over the Moraca river in Podgorica, University of Montenegro, Podgorica.

MIN (1969). Design papers of pedestrian bridge "Gazela", Nis.

Institute or material testing SRS. (1970). Report on the bridge "Gazela" testing, Belgrade.

Official Bulletin of SFRJ 1/91. (1991). Codes of practice on technical standards for determination of bridge loading, Belgrade.

HBM. (date unknown). Vibration measuring Unit (to DIN 45666 and DIN 456669 - FANAK) - Operating manual, Germany.

Vlajic Ljubomir. (1980). A contribution to analysis of dynamic behavior diagrams from theoretical vs. experimental point of view, JDGK, Trogir.