Dynamic tests and monitoring of a highway bridge over the Danube

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Jan BENCAT
Prof.
Univ. of Zilina
Zilina, Slovak Republic

Jan Bencat, born 1943, received his eng. degree in 1967 form TU Cracovia (Poland) and Ph.D. at UTC Zilina in 1973. He is currently Prof. of Structural Mechanics at Civil Eng. Faculty, Univ. of Zilina.

Summary

The dynamic response behaviour of a prestressed concrete seven span highway bridge (761,0 m long) was examined in September 1990 as a part of the static and dynamic loading tests (DLT)of the bridge. In this investigation, a structural measuring technique using vehicle-induced vibrations as well as forced vibrations induced by the rocket engines was developed for fullscale testing of the bridges. The data yielded the dynamic characteristics of the bridge e. g. natural frequencies f(j), mode shapes, dynamic load factor δ (DLF) and the damping of the structure (ϑ). The obtained dynamic characteristics were compared with the theoretical computed data. Monitoring of the highway bridge over the Danube (La Franconi Bridge) has been carried out in 1991-1997, to evaluate the accuracy when using a simple measurement of a well defined eigenfrequency to give a long term overall indication of deterioration or crack formation.

1. The Bridge Arrangement and Dynamic Loading Test

The main bridge structure is composed of seven span continuous beams with one frame pier (P3). Other supports are formed by seven massive piers. The total length of the bridge is 761.0 m with spans 83.0 m + 174.0 m + 172.0 m + 4 x 83.0 m. The highway bridge consists of two independent bridges (left and right bridge -LB,RB) with three traffic lanes, on each bridge for one direction only and sidewalks on both sides. The bridge box cross-section is shown in Fig. 1 and the longitudinal section, in Fig. 2. The bridge including multispan junctions is fully described in [1].
*Testing procedure and experimental analysis.*The test programme included field measurements using the instrumentation described in [3] so as to ensure coverage of entire possible range of vibration. The vibration amplitudes were investigated and recorded in 18 selected points. The time history of vertical as well as horizontal vibration has been registered by accelerometers (Brüel-Kjaer, BK-8306) in the 2nd and the 3rd span of the bridge (A1-A8) and in the other spans by inductive displacement transducers (IDT, range ±40 mm), points R1- R10, see Fig.1 and Fig.2. Output signals from the accelerometers were preamplified and recorded on two four-channel portable tape FM recorder (BK-7005) and simultaneously via DISYS software on PC/486 at the measuring station DMS-1. The signals from the IDT were recorded simultaneously at the station DSM-2 and DSM-3 by the same way as the signal from accelerometers. The experimental analysis has been carried out in the laboratory of the Department of Structural Mechanics UTC Žilina.

The dynamic load factor δ_{OBS} and frequency response spectrum (PSD) has been determined using the record obtained from passing vehicle velocities over the bridge by computer PC/486 via DISYS programme and two-channel real time analyzer BK-2032. *Testing load and experimental results.* The use of the test load has been in accordance with [3]. Two lorries TATRA-815 of mass 26 660 kg and 26 740 kg were used for the highway pavement. Rocket engines (RE) impulsive load were separately used, too. Static loading test (SLT) was perfomed before the

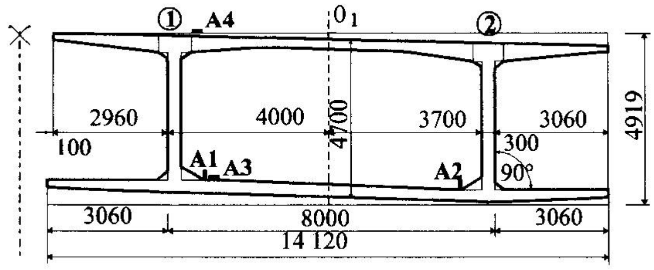


Fig.1 Bridge cross section

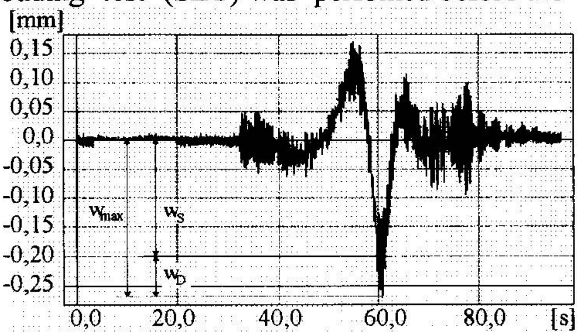
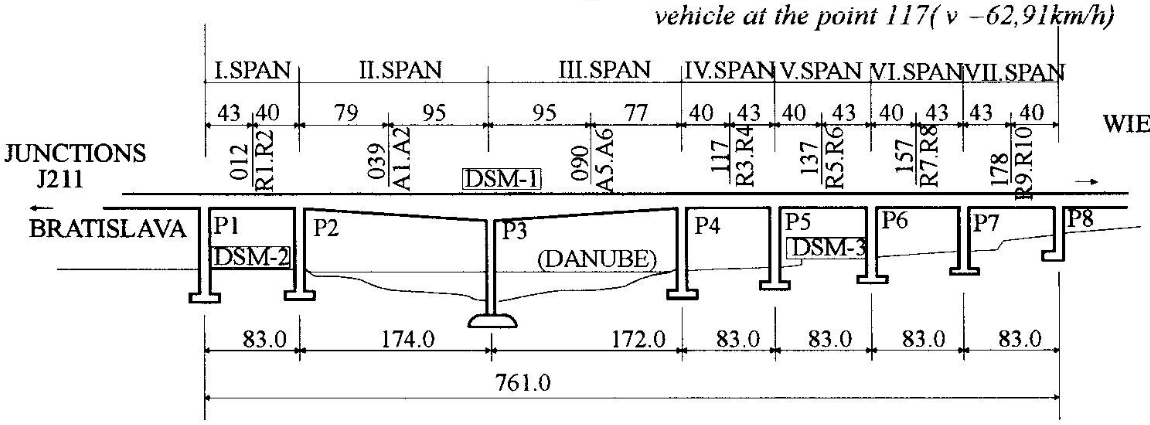


Fig.3 Time history of bridge vibration due to passing vehicle at the point 117(v –62,91km/h)



R1...R10 - Transducer Positions, A1...A6 - Accelerometer Positions

Fig.2 Longitudinal section of the bridge

dynamic loading test (DLT) with both vehicles TATRA-815. Experimental procedures have been discussed in detail by autor [3]. Formula (1) gives the criteria [2] for evaluation of moving load dynamic effects on the bridge structure

$$(\delta_{OBS} - 1)\eta_{DYN} \leq (\delta - 1) \tag{1}$$

where (see also Fig. 3), $\delta_{OBS} = w_{MAX} / w_S$ and $\eta_{DYN} = w_{DYN} / w$ The vertical static displacements at characteristic joint (012-178) due to standard live load (w) and due to testing load (w_{DYN}) have been computed by the bridge designer [1]. The calculated dynamic effectiveness η_{DYN} with theoretical and experimental values of the vertical displacements varied in the range η_{DYN} =0,09 ÷ 0,154, for different span of the bridge. The dynamic load factor δ (DLF) used by the designer for the bridge under investigation according to [4] is δ = 1.10, see also [1]. DLF of the experimental tests [3] are obtained from formula $\delta_{EXP} = 1 + (\delta_{OBS} - 1)\eta_{DYN}$, see also Fig. 3. The DLF against vehicle velocity are plotted in Fig. 4, 5. Natural frequencies have been obtained by using the spectral analysis from recorded responses due to various types of dynamic loading, see also Table 1. Since the accelerometers recorded only the dynamic component of vibrating structures, so we can consider those signals as an ergodic and stationary. The results of the analysis are fully described in [3]. Only two PSD are shown in Fig. 6, 7 of this paper. The comparison of theoretical (FEM) and experimental results of the natural frequencies *f(j)* of the vibration bridge according to [3] are presented in Table 1.

The spectral analysis of the vibration time histories made it possible to ascertain the dominating frequencies of bridge vibration by sharp peaks plotted in power spectra. The character of the vibration caused is heavily dependent on dynamic response excitation.

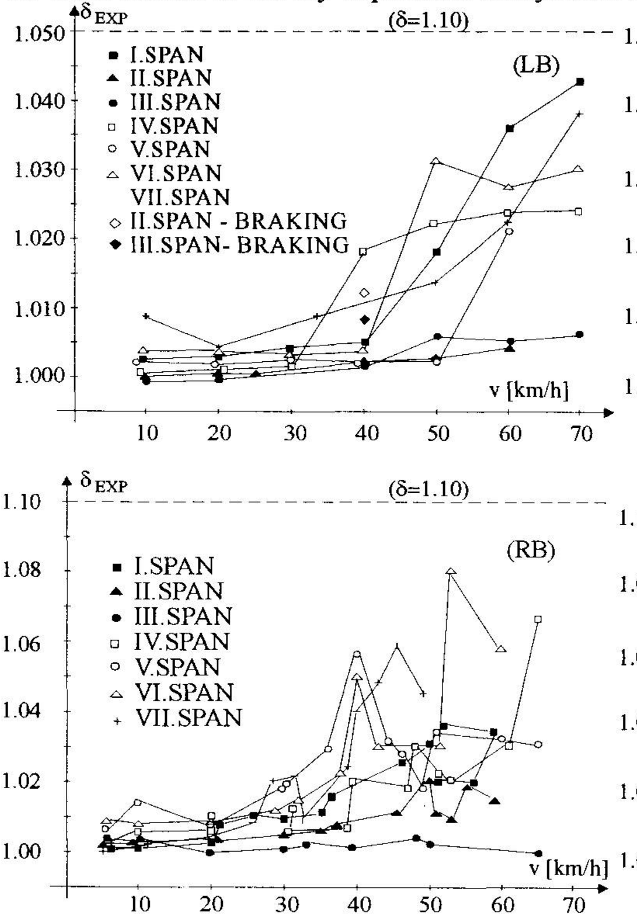


Fig. 4 DLF δ_{EXP} against vehicle velocity - vehicle smooth passing

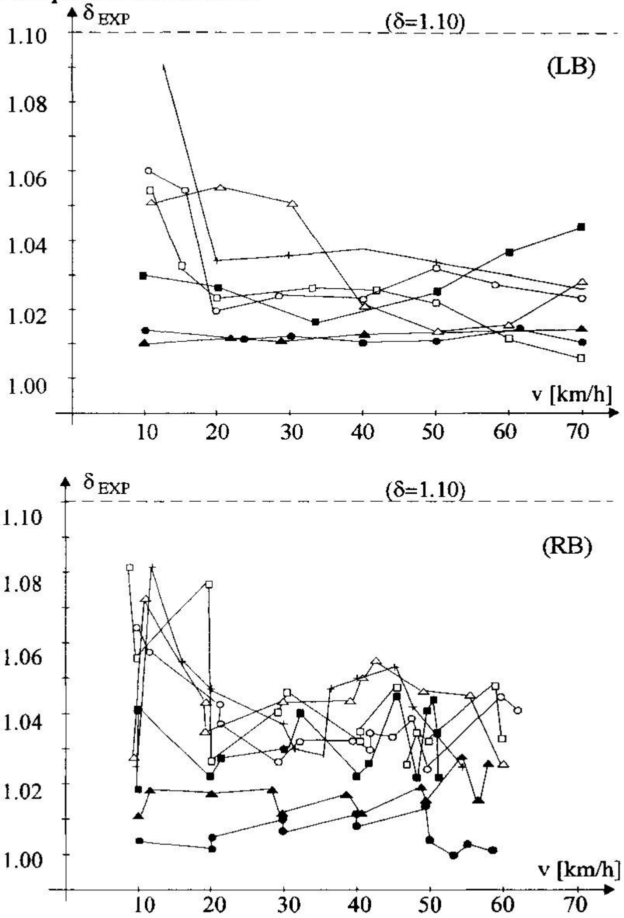


Fig. 5 DLF δ_{EXP} against vehicle velocity - vehicle obstacle passing

The results of the correlation analysis [3] showed that the vibration of the bridge is not ambient vibration. The bridge vibration has predominate vertical components (bending vibration). In this tests it was possible to predict the damping characteristics according to [2] by using logarithmic decrement.The evaluation of the logarithmic decrement ϑ has been done from records of fre bridge vibration due to impulse rocket engines. The logarithmic decrement corresponding to the first and second modes of the bridge vibrations varies in range ϑ=0,024 ÷ 0,049.

3.Bridge Dynamic Parameters Monitoring

Progressive deterioration of concrete structures (RC) due to alkali silica reactions and frost-thaw influence has become a serious problem. It has increased the importance of making observation on full scalle structures in order to obtain the experimental results necessary for the development of theories for predicting service life.

It has been the scope of this work to evaluate whether the relative change of a well defined natural frequencie or the change of the corresponding damping and the change of RMS value of the displacements amplitude of the bridge vibration observed by traffic loading can be used to give an overall indication of deterioration or crack formation. The monitoring technique based on measurement of the time history of the bridge vibration due to regular traffic is not meant to give detailed information but to be a technique simple to use to deside whether more detailed methods should be used.

During the years 1991-1997 La Franconi bridge over the Danube has been investigated by 24 hours monitoring tests in the summer and the winter time . A theoretical prediction of the bridge behaviour and preliminary dynamic loading tests are reported in [3].

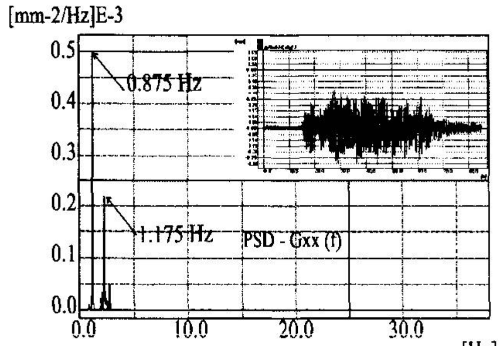


Fig.6 PSD at point 039 caused by TATRA 815

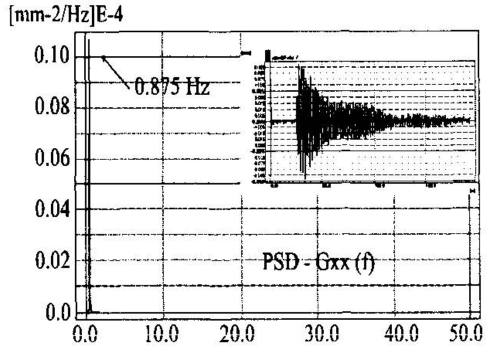


Fig.7 PSD at point 039 caused by RE

NATURAL FREQUENCIES / Hz /			EXCITATION TYPE
f(j)	CALCULATED	MEASURED	
1	0,865	0,875	RE, TATRA 815 - BR
2	1,157	1,175	TATRA 815 - OBST
3	1,919	1,9	RE
4	2,042	2,148	TATRA 815
5	2,3	2,275	RE, TATRA 815 - OBST
6	2,686	2,775	RE, TATRA 815 - BR

Table 1 The natural frequencies

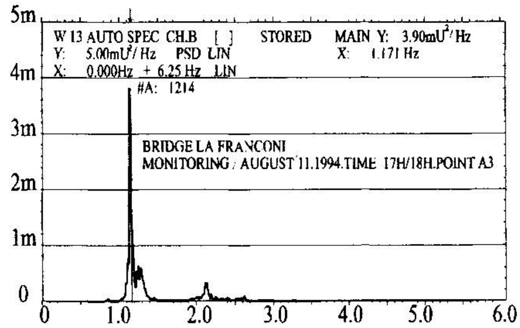


Fig. 8 Monitoring PSD at point 039

Testing procedure and experimental analysis. The vibration amplitudes were investigated and recorded in selected points of the second and the third span, see Fig. 1, 2. The time history of vertical vibration has been registered by accelerometers (Brüel-Kjaer, BK-8306) at points 039 and 090 (Al, A2, A5, A6) on each independent bridge.

Output signals from accelerometers were preamplified and recorded on four-channel analogue FM recorder BK-7005 and simultaneously via A/D convertor DAS-16 on portable notebook computer (PC/486) with special software (DISYS) and hardware facilities for 24 hours continuing test. The records obtained in the bridge monitoring tests were investigated by using frequency analyser BK-2034 and mentioned PC facilities. Fig. 8 shows power spectral densities (PSD) as an example of the spectral analysis of the monitoring test performed in August, 1994. The damping parameter (D-critical damping coefficient) was found by means of the 3dB bandwith method and curve fitting techniques. The amplitude analysis has been used to obtain RMS amplitude value of the bridge vibrations during the monitoring tests.

Experimental results. Results giving frequency and damping for lovest natural frequency in bending and RMS amplitude value from the monitoring tests of the bridges during whole measuring period are shown in Fig. 9. A 2.7% change in frequency is observed during a year (summer-winter) but it is systematic from one year to the next and is partly due to changes in ambient temperature. By measuring the frequency at the same time of year the changes from year to year are small and non systematic and correspond to a coefficient of variation of about 0.01. This may be considered negligible compared with the changes in natural frequency of about 30% corresponding to advanced deterioration observed in [5].

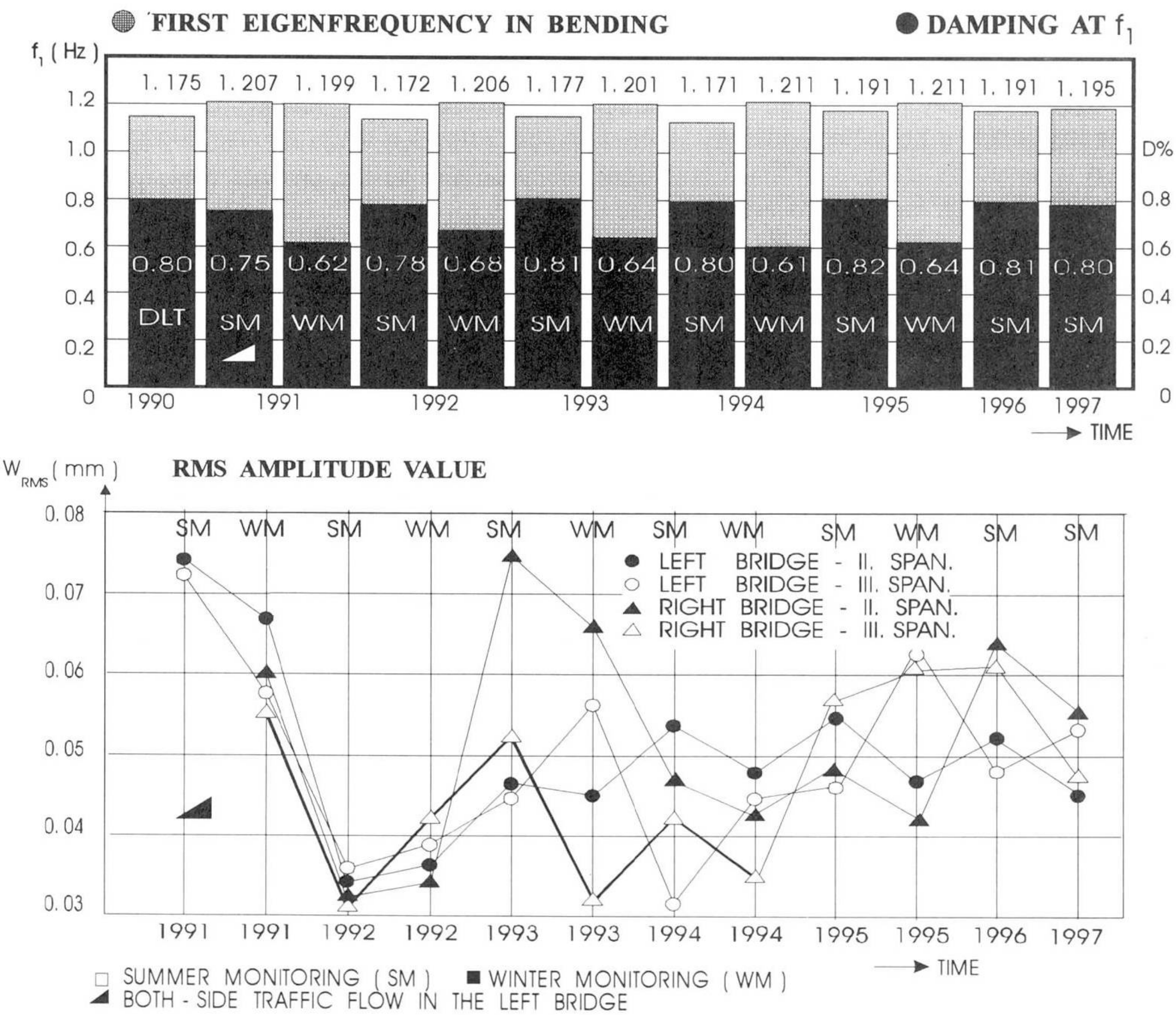


Fig.9 Changes in relative frequency, damping and displacement RMS amplitude values, 1990 - 1997

There is not the same systematic change of damping and scattering of results is big. What causes these changes is not clarified. There are changes in the temperature during the day. This may give changes in length of bridge which can influence support conditions and damping. Windspeed, water level, ambient relative humidity and temperature gradients through the deck, transport in the bridge deck, in particular at the surface, and may there by also change damping [6],[7].

There is a difference of the discplacement amplitude RMS value measured in may 1991 in comparison with other measurements results. It was maybe caused by both-side motor traffic flows on the left bridge. All the following measurements were performed in conditions of the one side traffic flow on each of the two bridges La Franconi. The changes of the amplitude RMS value is caused mainly by changes of the intensity of the regular motor traffic.

4. Conclusion

The experimental analysis of the bridge dynamic response caused by moving load as well as impulsive load made it possible to identify six basic modes of bridge vibration. These frequencies have been received by analysis of small amplitude vibration and so the analysis corresponds to linear vibration. It was possible to evaluated the damping characteristics of the bridge structure only from limited number of measurements. They are therefore only indicative. The experimentally achieved dynamic load factor δ_{EXP}<1,09 shows that real stiffness of structure is fully comparable with the corresponding value for, δ = 1,10 obtained by computation. The comparison of theoretical and experimental results of the bridge parameters shows good agreement of theoretical and experimental values of natural frequencies. The criteria of all loading states by the Slovak standard [2] are seen to have been fulfilled.

The monitoring tests results show that the relative change of a well defined natural frequency seems to be very little influenced by changes in temperature, humidity, support conditions, etc., in fully hardened not deteriorated RC structure of simple geometry, if measurements to be compared are made at the same time of the year. This indicates that the monitoring tests may prove useful by giving an idea of the overall development of long term deterioration and cracking in RC structures. The change in structural damping can so far not be used in a similar way because of its big dependance on mentioned secondary influences which are comparable with deterioration or cracking influences on change of structural damping.

The changes in deflection amplitude RMS value are heavily dependent on intensity of regular motor traffic in the bridge deck, but vary within of the 50% RMS amplitude range measured during the whole measuring period.

5. Acknowledgment

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