HIGHWAY BRIDGES DYNAMIC PARAMETERS MONITORING

Ján Benèat

ABSTRACT

Experimental monitoring tests have shown that an early indication of deterioraton seems possible by observing the relative change of eigenfrequencies. Additional informations in this field can be gained by observing the relative change of the damping and effective (RMS) amplitude of the bridges vibrations due to regular traffic. 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. It is concluded that the monitoring should be considered as a tool when evaluating structural integrity in RC structures.

1. Introduction

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 frequencies or the change of the corresponding damping and the change of RMS value of the 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 decide 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 [8,9]. A theoretical prediction of the bridge behaviour and preliminary dynamic loading tests are reported in [10].

2. The bridge arrangement

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 83.0m + 174.0m + 172.0m + 4x83.0m. The highway bridge consists of two independent bridges (left and right bridge) with three traffic lanes each (e.g. three in 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**[10].The bridge multispan junction is fully described in [10]

3. Testing procedure and experimental analysis

The test programme included field measurements using the instrumentation described in [10] so as to ensure coverage of entire possible range of vibration. 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 A1,A2,A5,A6.

Output signals from accelerometers were preamfilied and recorded on four-channel analogue BK-7005 and simultaneously via A/D convertor DAS-16 on portable notebook computer (PC/486) with special software and hardware facilities for 24 hours continuing test.

The experimental analysis has been caried out in the laboratory of the Department of Structural Mechanics U.T.C. •ilina. The records obtained in the bridge monitoring tests were investigated by using frequency analyser BK 2034 and mentioned PC facilities. **Fig. 3** 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.

The measuring system was calibrated occasionally during the measuring period and inaccuracy of the frequency in the actual frequency range corresponded to a coefficient of variation less than 0.01. It was also found that the recording was insensitive to changes in the ambient temperature, which was measured.

4. 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. 4**. 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 the year the changes from the year to the 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 [4]. 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 there by damping. Windspeed, water level, ambient relative humidity and temperature and temperature gradients through the deck, transport in the bridge deck, in particular at the surface, and may there by also change damping. [6]

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 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.

5.Conclusion

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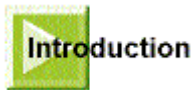
The changes in deflection amplitude RMS value are heavily dependent on intensity of regular motor traffic in the bridge deck, but vary within of the 50 % RMS amplitude range measured during the whole measuring period.

6.Acknowledgment

7.References

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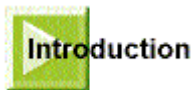
AUTHOR BIOGRAPHIES

Ján Benèat

Professor

Civil Enginnering Faculty

University of •ilina

•ilina, Slovakia

Born in 1943. Jan Benèat received his civil engineering degree in 1967 from TU Cracovia (Poland). He completed his Ph.D. at University of Transport and Communications in •ilina (UTC •ilina) in 1973. He is currently professor of Structural Mechanics at CEF University of •ilina. He has published widely in a variety of research domains including structural dynamic, dynamic of bridges, soil dynamic and microtremor due to transport.
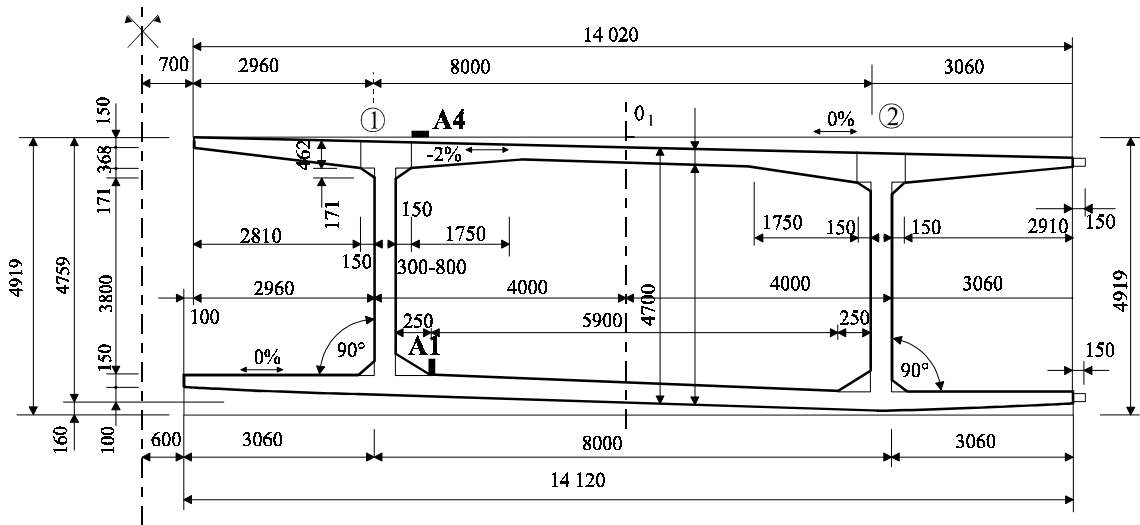
Fig. 1. Cross Section of the Bridge

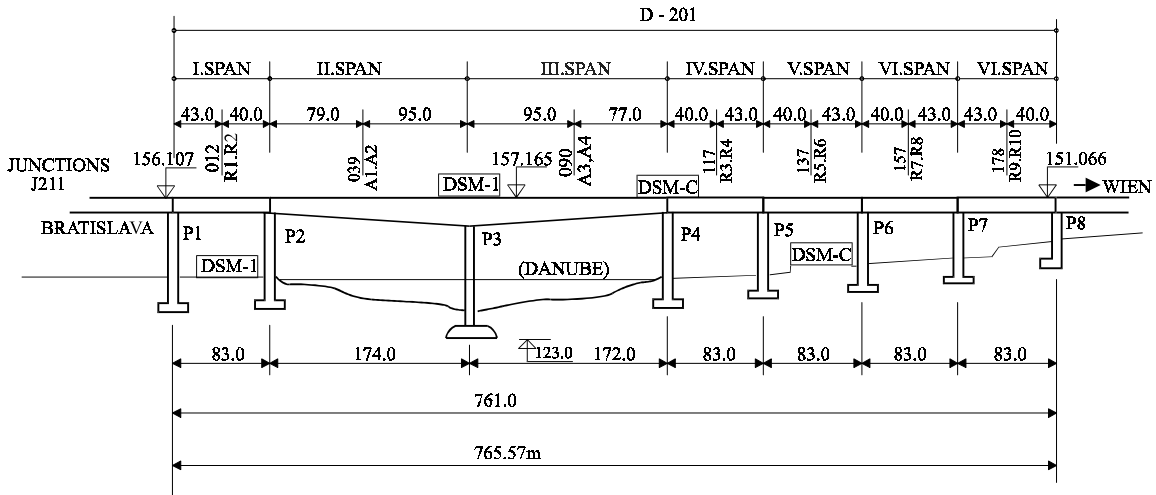Return to paper

Content

Fig. 2. Longitudinal Section of the Bridge

DSM - 1,2,3, - DEPARTMENT OF STRUCTURAL MECHANICS MEASURING STATION
DSM - C   - DEPARTMENT OF STRUCTURAL MECHANICS MESURING HEADQUARTER
A1...A6   - ACCELEROMETERS POSITION

W 13 AUTO SPEC CH.B [ ]   STORED   MAIN Y: 3.90mU²/Hz
Y: 5.00mU²/Hz   PSD LIN   X: 1.171 Hz
X: 0.000Hz + 6.25 Hz LIN
#A: 1214

BRIDGE LA FRANCONI
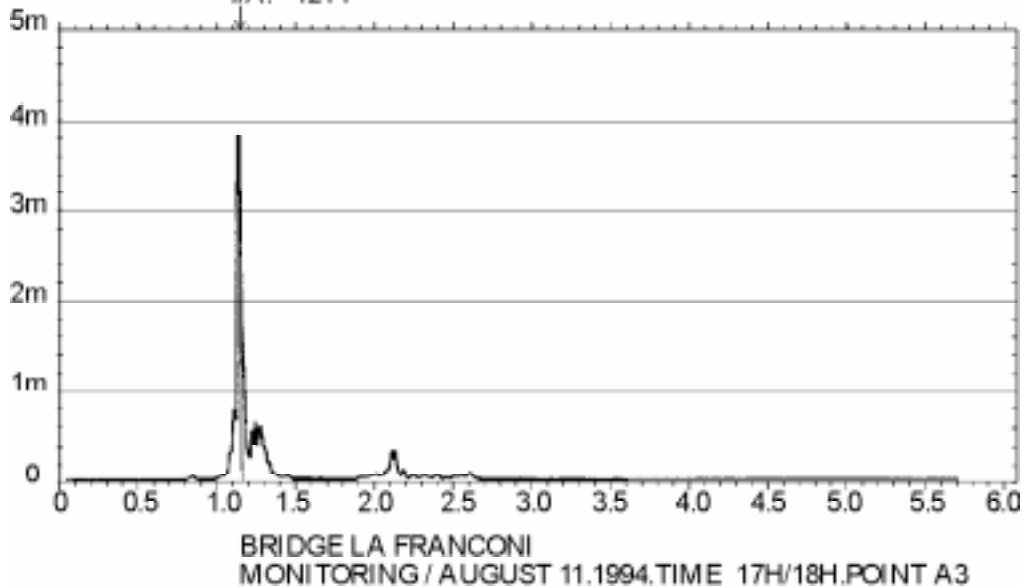MONITORING / AUGUST 11.1994.TIME 17H/18H.POINT A3

Fig. 3. Power Spectral Densities of the bridge Vibration Displacement
Amplitude at Point A3

Return
to paper

Content

FIRST EIGENFREQUENCY IN BENDING

f_1 (Hz)

1.175	1.207	1.199	1.172	1.206	1.177	1.201	1.171	1.211	1.191	1.211	1.191	1.195
/ DLT /	SM	WM	SM	WM	SM	WM	SM	WM	SM	WM	SM	SM

1990 1991 1992 1993 1994 1995 1996 1997

→ TIME

DAMPING AT f_1

D (%)

0.80	0.75	0.62	0.78	0.68	0.81	0.64	0.80	0.61	0.82	0.64	0.81	0.80
/ DLT /	SM	WM	SM	WM	SM	WM	SM	WM	SM	WM	SM	SM

1990 1991 1992 1993 1994 1995 1996 1997

→ TIME

RMS AMPLITUDE VALUE

W_{RMS} (mm)

SM WM SM WM SM WM SM WM SM WM SM SM

● LEFT BRIDGE - II. SPAN.
○ LEFT BRIDGE - III. SPAN.
▲ RIGHT BRIDGE - II. SPAN.
△ RIGHT BRIDGE - III. SPAN.

1991 1991 1992 1992 1993 1993 1994 1994 1995 1995 1996 1997

→ TIME

□ SUMMER MONITORING (SM) ■ WINTER MONITORING (WM)
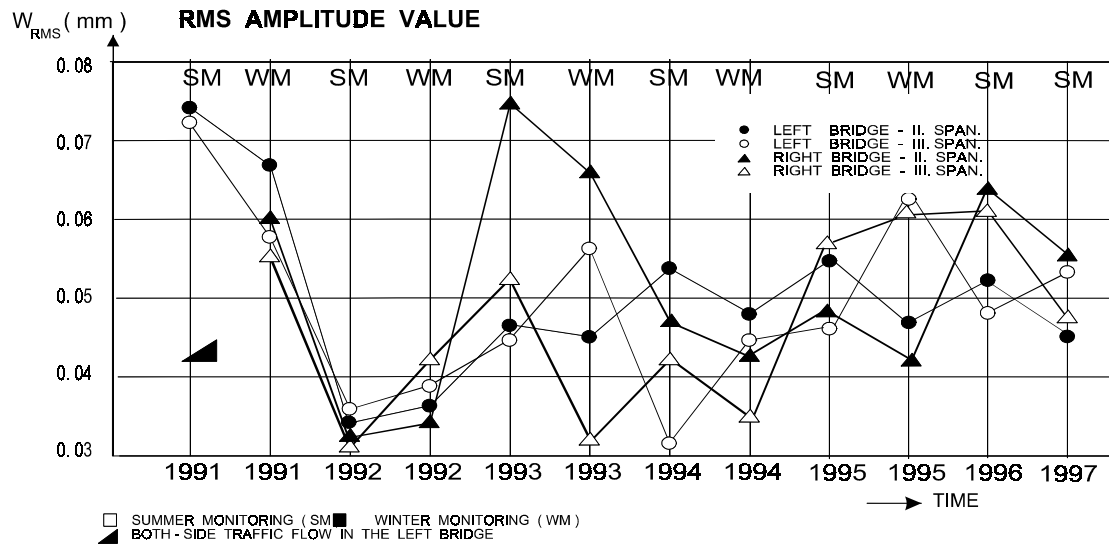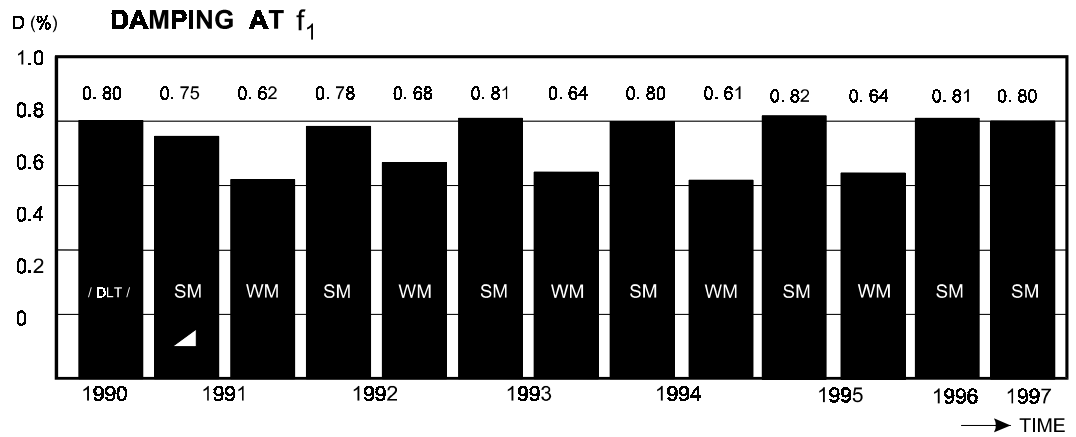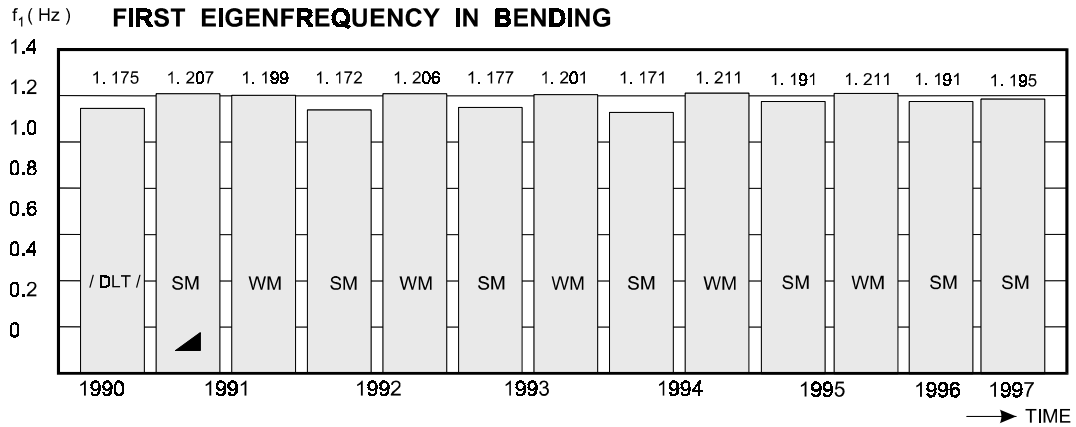◤ BOTH - SIDE TRAFFIC FLOW IN THE LEFT BRIDGE

Fig.4 Changes in Relative Frequency, Damping and Displacement RMS Amplitude Values, 1990 - 1997

Return to paper

Content