Multi-span highway bridge dynamic test and its dynamic response calculation

Dr J. SLAŠTAN, Associate Professor, University of Transport and Communications, Žilina, Czechoslovakia, and EMPA, Swiss Federal Laboratory of Materials Testing and Research, Duebendorf, Switzerland, and Dr J. MELCER, Associate Professor, University of Transport and Communications, Žilina, Czechoslovakia

The paper deals on dynamic load test of the nine-span higway bridge carried out before the opening of the bridge. The bridge behaviour was not fully in accordance with the designer's assumptions and the additional experimental investigation of two piers with two adjacent bridge spans was undertaken. The dynamic bridge response calculation as the bridge/lorries interaction has been made with taking into account the lorries speeds and the suspension characteristics, and the results of the abovementioned measurement of the elastic pier properties.

1. INTRODUCTION

A dynamic loading test was undertaken on the nine-span highway bridge to the according of Czecho-Slovak Standard (ref. 1) before the opening of the bridge.

The test has been carried out in order to identify the bridge dynamic characteristics (dynamic coefficient, natural frequencies, damping) using both lorries passing over the bridge with and without plank, and rocket motors.

From the tests, it was found that the bridge did not satisfy the criteria given by the Standard (ref. 1).

When a new bridge fails to satisfy the Standard, the following options are given for compliance:

a. Complete a dynamic calculation
b. Reduce the design capacity
c. Reconstruct the bridge.

The behaviour of the bridge piers was not fully in accordance with the designer's assumptions so it was decided to verify the real action experimentally. The parameters obtained from the tests were used in dynamic calculations.

2. DIMENSIONS OF STRUCTURE UNDER INVESTIGATION

The bridge investigated was a continuous nine-span (36.0 m + 7 x 70.0 m + 36.0 m) prestressed structure, Fig. 1, consisting of concrete box girder segments with dimensions given by Fig. 2 and mass of 8.5 t each of them.

The bridge was designed as the two independent structures but only one part had been completed in the time of investigation.

The bridge longitudinal axis is represented by a space curve changing from 0.6% (left support, I) to 3.41% (right support, X) with R = 2300 m in horizontal plane and with R = 20000 m in the vertical plane.

The each pier is composed of a pair of massive concrete walls spaced at 5.0 m in the longitudinal direction. The cross section of each wall is 1.2 m x x 6.0 m.The pier heights vary from 11.0 m to 27.0 m. The assumed boundary conditions are hinged - clamped (piers III - VII) and hinged - hinged (piers II, VIII, IX). A stiffening concrete element between the walls of the pier V improves longitudinal stiffness. Pier V resist most of the longitudinal force. The pier foundations comprise concrete footings supported by piles.

3. DYNAMIC COEFFICIENT

The basic criterion given by Standard (ref. 1) is shown as equation (1)

$$(\delta - 1).\eta \leq (\delta^* - 1) \qquad (1)$$

where

δ is observed dynamic coefficient
δ^* is dynamic coefficient used at the bridge design.

Equation (1) should be fulfilled at least in 90% of all tests of vehicles over the bridge. The rest of the tests should be in accordance with

$$(\delta - 1).\eta \leq 1.1(\delta^* - 1) \qquad (2)$$

In equations (1) and (2), a dynamic effectiveness of loading vehicle, η, is defined as

$$\eta = \frac{S_{dyn}}{S} \qquad (3)$$

where

S_{dyn} is calculated value of static deflection of the measured bridge span caused by the test load for the dynamic test
S is calculated deflection at the same span caused by design live load.

The definition of observed dynamic coefficient, δ, is given by Fig. 3 from (ref. 1) as

Heavy vehicles and roads: technology, safety and policy. Thomas Telford, London, 1992.

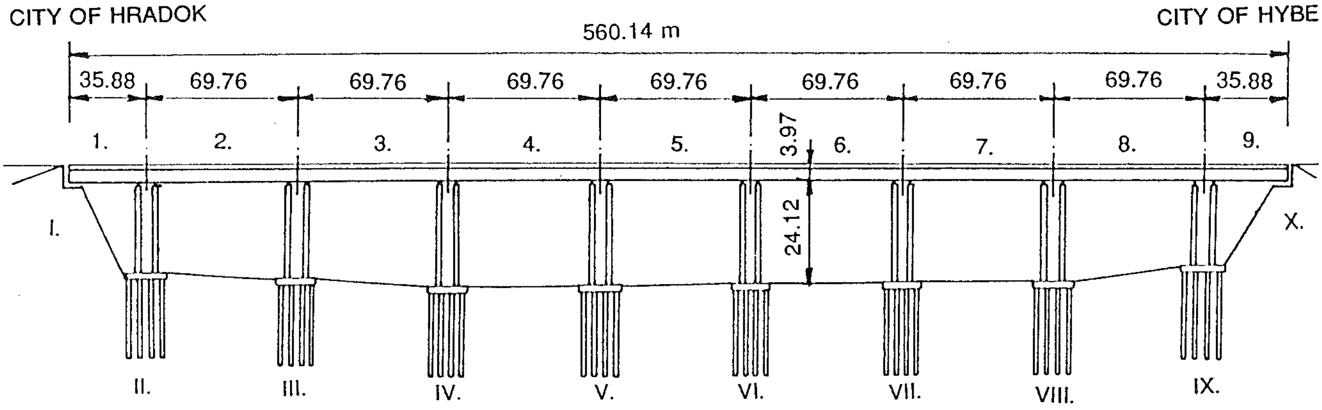


Fig. 1. The bridge elevation

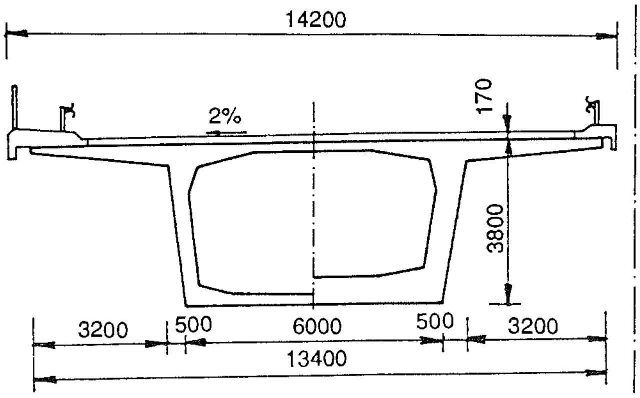


Fig. 2. The bridge cross-section

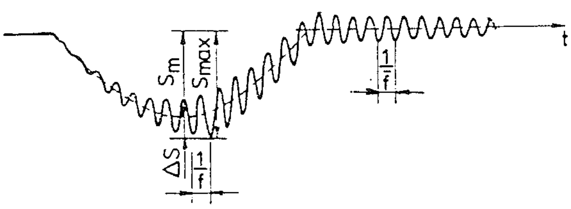


Fig. 3. Definition of dynamic coefficient

$$\delta = \frac{S_{max}}{S_m} \tag{4}$$

Where S_{max} and S_m are defined as shown in Fig. 3.

4. TESTING PROCEDURES

The dynamic coefficients are directly related to a static model of structure. A valid comparison between δ and standardized dynamic coefficient, δ^*, is only possible when the real behaviour of the investigated structure is close to assumptions used for calculation.

Criterion (1) was not fulfilled as prescribed by the Standard (ref. 1). A dynamic bridge calculation was necessary.

Because the results of standard dynamic as well as static loading tests (ref. 2) indicated certain differences between static model and real behaviour of structure, it was decided to examine real action of bridge piers and the two adjacent spans.

4.1 Static test

The aim of a static test was to identify the real action of piers with different assumed boundary conditions and to determine of the elastic properties for use in calculation.

The pier VI with the height of 24.12 m, and assumed hinged-clamped boundary conditions, and the pier IX of height 11.26 m with boundary conditions defined as hinged-hinged were chosen for the investigation.

The position of measuring points over the pier VI, foundation slab and bridge superstructure is given in Figs 4-6. The vertical displacements of midpoints of two adjacent spans to the pier were also measured. Pier IX was instrumented in the same way. The code L refers to a rotation and M vertical displacement measurement points.

The five load positions of 6 lorries, Fig. 7, each with a mass about 22.0 tonnes, have been used when testing pier VI. For static testing, three lanes of lorries were used. These were with the heavily loaded rear axles adjacent to each other. Similar arrangement of load positions used pier IX.

The results of static tests are summarized in Table 1. The results showed that the structure behaves as a nine-span continuous beam supported by eight rotational springs restraints each formed from a pair of concrete pier walls.

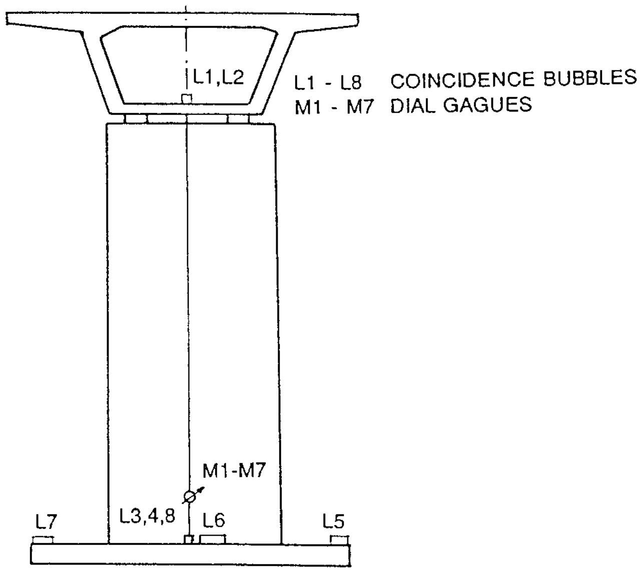


Fig. 4. Measured points over the pier VI cross-section

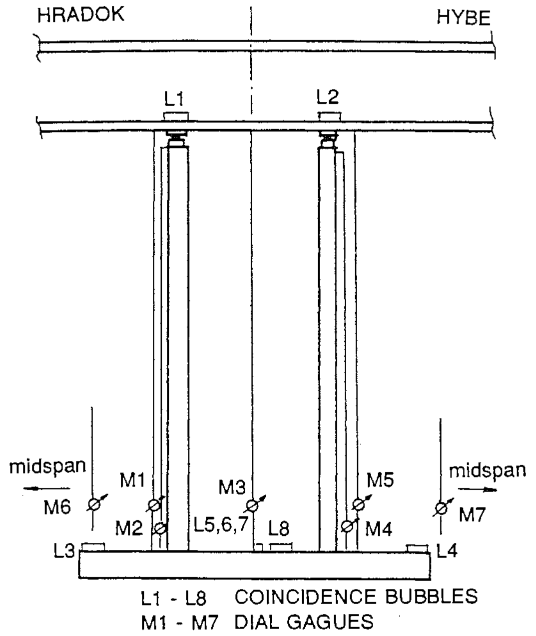


Fig. 5. Measured points over the pier VI elevation

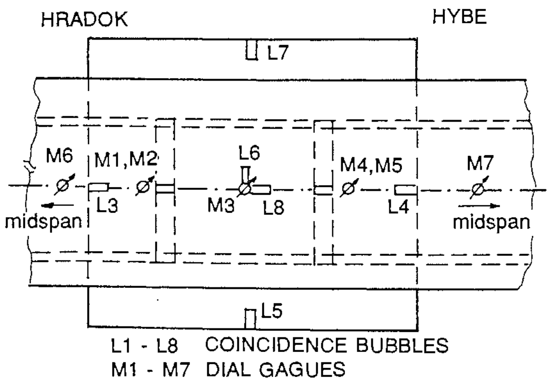


Fig. 6. Measured points at the pier VI top-view

Table 1. Measured displacements

Load positions		Vertical displacements [mm] M1	M2	M3	M4	M5	M6	M7
L.P. - 1	PVI	0.05	0.00	0.10	0.00	0.10	- 0.40	- 0.50
	PIX	-	-	-	-	-	-	-
L.P. - 2	PVI	0.30	0.05	0.00	0.00	0.00	0.40	0.00
	PIX	0.15	0.10	0.10	- 0.05	-0.10	0.70	-0.05
L.P. - 3	PVI	-0.30	-0.10	- 0.15	0.05	0.05	-0.20	0.60
	PIX	-	-	-	-	-	-	-
L.P. - 4	PVI	0.30	0.20	-0.10	- 0.05	-0.05	4.30	-0.70
	PIX	0.10	0.05	0.05	- 0.05	-0.15	3.60	-0.40
L.P. - 5	PVI	-0.30	- 0.30	-0.10	0.20	0.20	-0.80	4.40
	PIX	0.00	0.0	0.05	0.05	0.10	-0.30	1.00

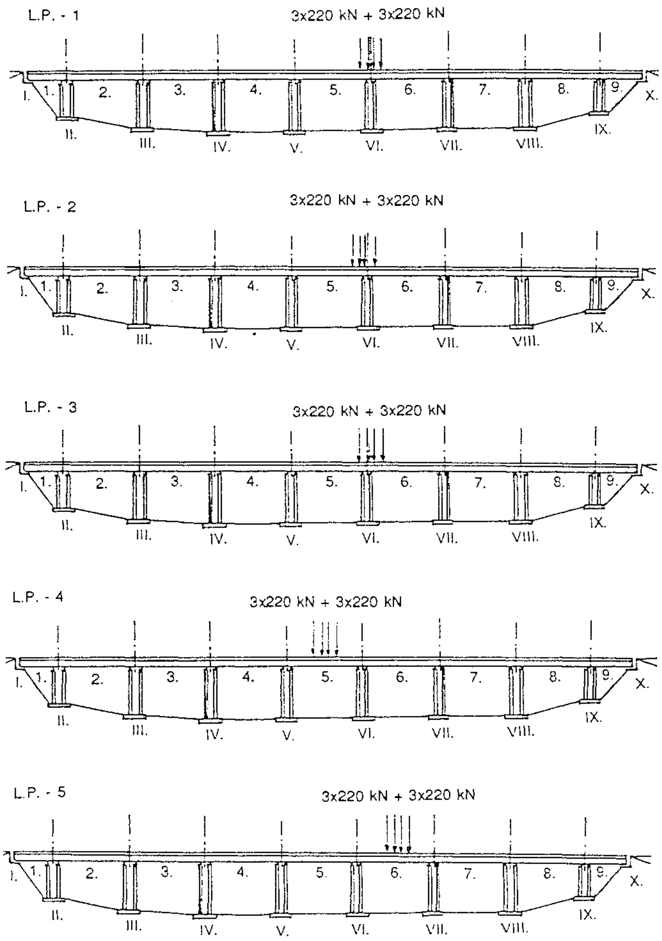


Fig. 7. Bridge load positions on the pier VI and adjacent spans

The calculation of mid-span bridge deflections were made using reversed model (Fig. 8). Comparison gives good agreement between theory and experiment, see Tables 2-3.

4.2 Dynamic test

Dynamic tests have been carried out to identify some dynamic characteristics of the pier walls, foundation slabs and adjacent bridge parts as well as verify of any interaction between various components.

Two trucks with mass of 22.0 tonnes, each moving side by side over the bridge, were used for the dynamic tests

The position and type of receivers used is shown in Figs 9-10.

A total of 24 tests with pairs of vehicles were applied to the bridge. The results with corresponding values of δ are given in Table 4.

The codes of vehicle runs give information on: part of structure tested, number of passing vehicles, directions of run over the bridge, position of plank, and desired vehicle speed. The first part of code: L-left part of bridge, 02-two vehicles. The second part of code: 1-runs in direction Hradok - Hybe, or 2 -opposite direction Hybe - Hradok, last two numbers of the second code part: 00-bridge deck without plank, 05-plank placed at the 5th bridge midspan, 06-plank positioned at the 6th bridge midspan etc.,

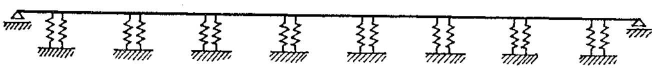


Fig. 8. Model for the bridge calculation

Table 2. Comparison of measured and calculated midspan deflection - 5th and 6th spans

Load position	S(5)-teor [mm]	S(5) - exp [mm]	S(6)-teor [mm]	S(6) - exp [mm]
L.P. - 1	-	-0.4000	-	-0.5000
L.P. - 2	0.3890	0.2500	-0.1108	-0.0500
L.P. - 3	-0.1104	-0.0500	0.3890	0.5750
L.P. - 4	4.2167	4.1500	-0.8438	-0.6750
L.P. - 5	-0.8438	-0.6500	4.2167	4.3004

Table 3. Comparison of measured and calculated midspan deflection - 8th and 9th spans

Load position	S(8)-teor [mm]	S(8) - exp [mm]	S(9)-teor [mm]	S(9) - exp [mm]
L.P. - 1	-	-	-	-
L.P. - 2	0.3631	0.5250	-0.0274	-0.0130
L.P. - 3	-	-	-	-
L.P. - 4	4.0900	3.6500	-0.4721	-0.4750
L.P. - 5	-0.4531	-0.3000	1.0770	0.9500

VI-plank over the VI-th pier, IX-plank over the IX-th pier.The last part of code represents a desired vehicle velocity.

The δ has been evaluated only for the midspan of the most critical bridge spans from the previous tests (ref. 2), 6th span (runs no. 1-12) and 8th (runs no.13-24) span.

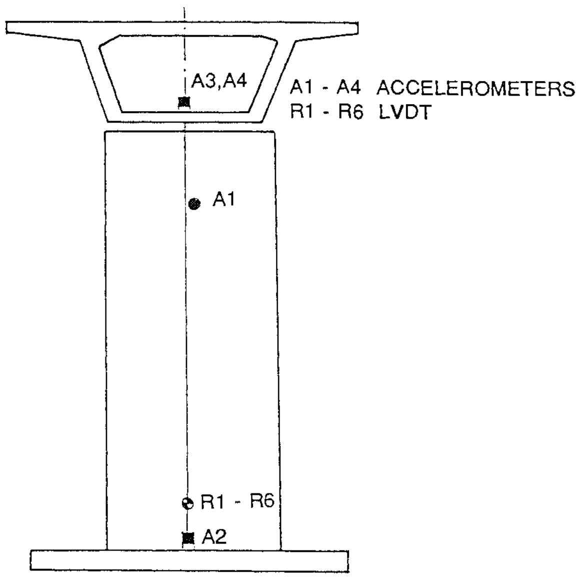


Fig. 9. Measured points over the pier VI cross-section - dynamic test

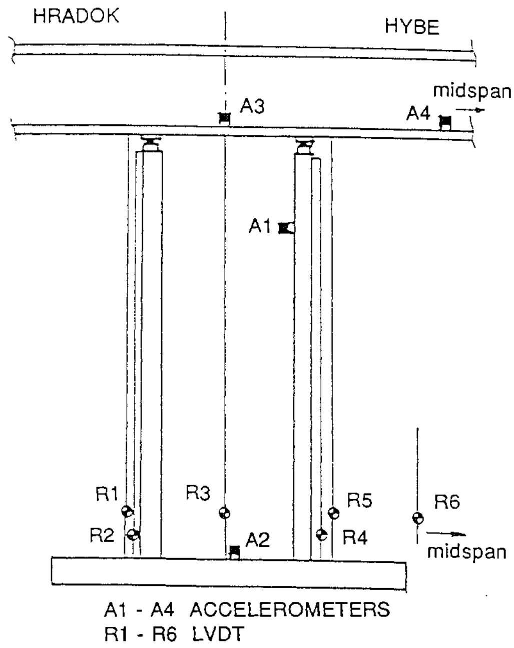


Fig. 10. Measured points over the pier VI elevation - dynamic test

The dynamic effectiveness of loading vehicles for reduction of δ in according of Czecho-Slovak standard (ref. 1) has been calculated as $\eta(6)$ = =0.26709 and $\eta(8)$ = 0.27151 for corresponding bridge spans.

Table 4. Dynamic coefficients

No.of run	Code of run	δ	$(\delta-1)\eta+1$	v [km/h]
1	L02 2VI 10	1.12018	1.03209	17.20
2	L02 2VI 40	1.16880	1.04509	31.73
3	L02 2VI 60	1.29629	1.07900	38.18
4	L02 105 10	1.10170	1.02716	21.35
5	L02 105 40	1.04760	1.01272	26.25
6	L02 105 60	1.33333	1.08903	37.05
7	L02 206 10	1.23711	1.06333	23.35
8	L02 206 40	1.36842	1.09841	27.39
9	L02 206 60	1.39534	1.10559	57.27
10	L02 106 10	1.11111	1.02967	23.77
11	L02 106 40	1.23711	1.06333	26.25
12	L02 106 60	1.49944	1.13339	34.05
13	L02 1IX 10	1.09091	1.02468	13.40
14	L02 1IX 40	1.25000	1.06788	33.15
15	L02 1IX 60	1.26316	1.07145	57.27
16	L02 208 10	1.54322	1.14749	17.50
17	L02 208 40	1.20000	1.05430	26.80
18	L02 208 60	1.55173	1.14980	54.78
19	L02 109 10	1.13850	1.03634	11.60
20	L02 109 40	1.14943	1.04057	33.15
21	L02 109 60	1.44927	1.12198	38.18
22	L02 200 10	1.20482	1.05561	14.15
23	L02 200 20	1.29870	1.08110	22.62
24	L02 200 30	1.25000	1.06787	28.18

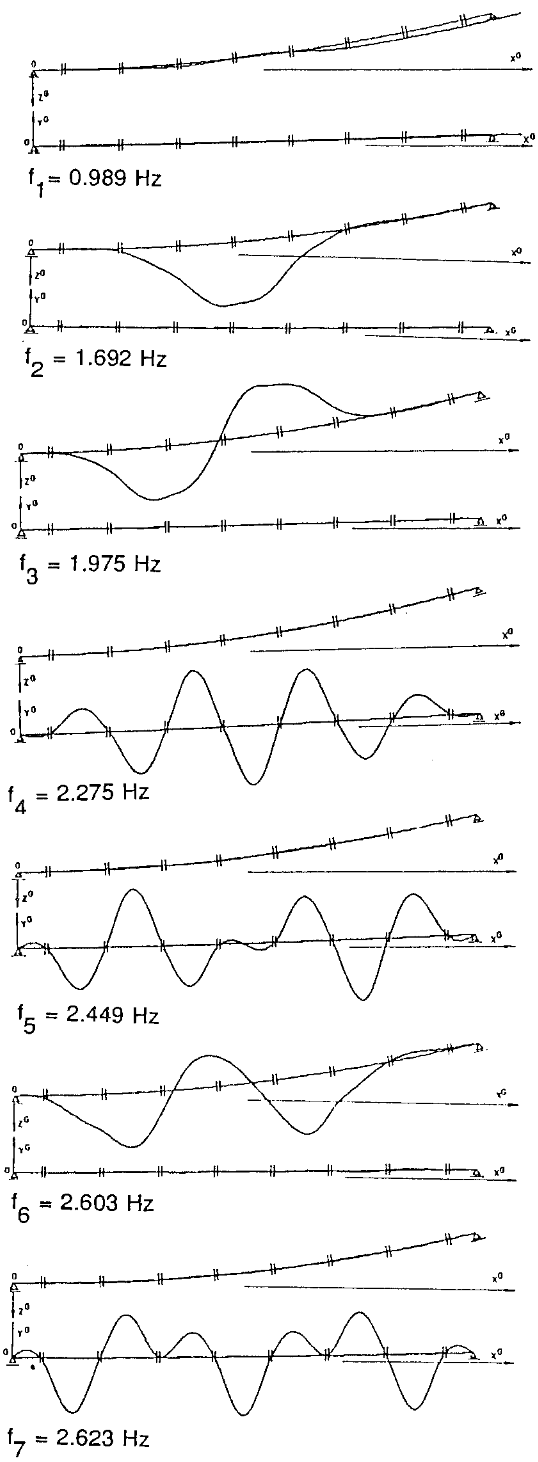
5. DYNAMIC CALCULATION

5.1 Natural frequencies

Calculation of natural frequencies of the bridge vibrations has been done by FEM taking into account geometry of structure, and elastic properties of piers in vertical and both horizontal directions. An assumption of torsionally solid segments over supports has been included into bridge simulation model.

The first ten modes of bridge superstructure vibration in horizontal and vertical planes as well of its longitudinal vibration are shown in Fig. 11.

The comparison of calculated and measured frequencies is shown in Table 5.



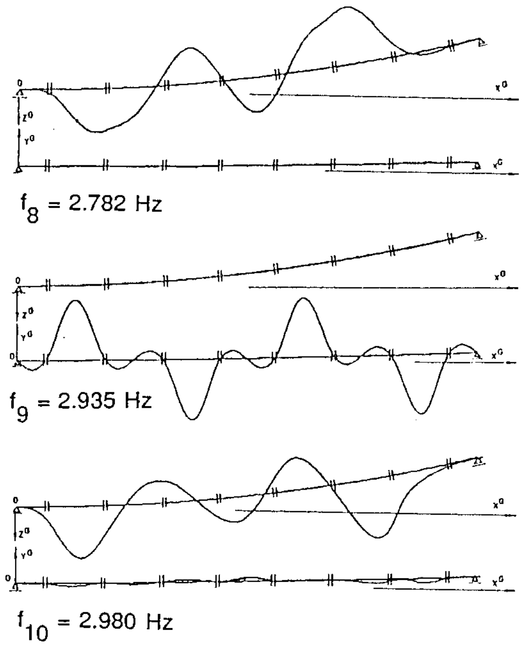


Fig. 11. Modes of the bridge vibration with corresponding natural frequencies

Table 5. Comparison of calculated and measured frequencies

Mode	f[Hz] teor	f[Hz] exp
1	0.989	1.0-1.05
2	1.692	1.5-1.63
3	1.975	1.8-1.87
4	2.275	2.2-2.37
5	2.449	2.4-2.50
6	2.603	2.600
7	2.623	2.62-2.63
8	2.782	2.7-2.80
9	2.935	2.875
10	2.980	3.0-3.5

5.2. Dynamic coefficient

The dynamic coefficients δ, presented in this paper, have been determined from the bridge response induced by simultaneous runs of two vehicles arranged in one row, as they cross over a plank placed at the 6th bridge midspan.

The motion equations describing of synchronous vibration of the system vehicle-bridge have been derived in form of the second-order differential Lagrange's equation

$$\frac{d}{dt}\left[\frac{\partial T(t)}{\partial \left(\frac{dq}{dt}\right)}\right] - \frac{\partial T(t)}{\partial q} + \frac{\partial V(t)}{\partial q} + \frac{\partial D(t)}{\partial \left(\frac{dq}{dt}\right)} = 0$$

where

a(t), z(t), φ(t) have been stepwise substituted in the generalized coordinate, q

V(t) is potencial energy of system

T(t) is kinetic energy of system

D(t) is absorbed energy in vehicle absorber and bridge in time unit

φ(t) is rotation angle of susupended vehicle mass in vertical plane around of centroid

z(t) is vertical amplitude vibration of the suspended vehicle mass centroid

a(t) is proportionality coefficient

The coefficient a (t) is based on the following:

- vehicle is plane model, Fig. 12
- Bernouli-Euler's beam model with only linear deformations
- vehicle suspensions elements are linear elastic
- vehicle absorbers have viscous damping
- damping is proportional to the vehicle vibration velocity

elastic and damping vehicle tires properties are neglected

vehicles move with constant speed

dynamic bending line of beam axis is proportional to bending line caused by a static effect (see Fig. 13)

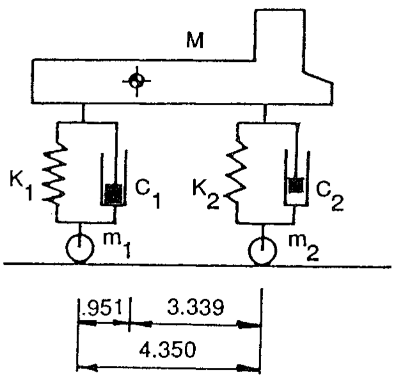
vehicle is always in contact with bridge.

The system of equations has been solved by using of Merson's modification of classical numerical integration method of Runge-Kutta.

The dynamic coefficient has been obtained as the ratio between dynamic amplitude S_{max} induced by a passing testing trucks over a bridge and static deflection S_m caused by these vehicles at the same bridge midspan.

The calculation of the dynamic midspan deflection has been performed with vehicles parameters (mass, suspension characteristics, speeds) which corresponds to the values used during the bridge test.

The dynamic coefficients calculated and observed from the tests as a function of vehicle speed are plotted in Fig. 14.



m_1 = 2 120 kg

m_2 = 880 kg

M = 19 000 kg

C_1 = 260 197 Ns/m

C_2 = 72 800 Ns/m

K_1 = 1.70617 MN/m

K_2 = 0.27992 MN/m

Fig. 12. Vehicle model

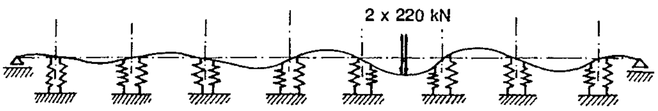


Fig. 13. Assumed bridge deformation

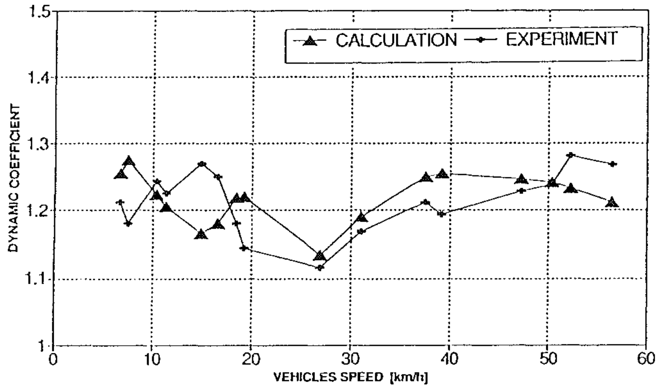


Fig. 14. Dynamic coefficient vs. vehicles speed

6. CONCLUSIONS

This paper briefly describes the procedure for testing of bridges to according of Czecho-Slovak Standard (ref. 1).

Additional experimental verification of real bridge behaviour allowed to proposed the new bridge model simulation for its dynamic calculation.

The simplifying assumptions introduced into calculation are acceptable only if distance between two forces (4.35 m) caused midspan deflections is small in comparison of bridge spans (70.0 m) and beam elements are deformed in assumed shape represented by the dominant mode of structure vibrations evaluated from spectral or modal analysis if is available.

It is necessary to mention that quality of bridge pavement surface which has also significant influence on magnitude of dynamic deflection (ref. 4) has been neglected. The information on the roadway surface quality has been not available at the time of experimentation.

The implementation of the effects of pavement bridge roughness quality described by power spectral density, into proposed simply model of vehicle-bridge interaction is in progress at the University of Transport and Communications in Žilina (Czecho-Slovakia).

7. ACKNOWLEDGEMENT

The authors appreciate the financial support given by Head Office of Highways in Bratislava allowing the additional experimental work.

REFERENCES

1. ČSN 73 6209 Loading tests of bridges, p. 32. Institute for Standardization and Measurments, Prague, 1987.
2. SLAŠTAN J. and BATA M. Report on the dynamic loading test of highway bridge Hybica. p. 153. University of Transport and Communications, Žilina, 1991.
3. SLAŠTAN J. Report on dynamic calculation of Hybica bridge with experimental evalution of inputs parameters. p. 138. Head Office of Highways, Bratislava, 1992.
4. CANTIENI R. Contribution to the dynamics of highway bridges under heavy traffic. p. 147. PhD. Thesis No.9505, Technical University of Zurich, 1991.