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DYNAMIC TESTING OF A MODERN CONCRETE BRIDGE

by J. H. Rainer and G. Pernica

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Dynamic testing of a modern concrete bridge

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A posttensioned reinforced concrete bridge, slated for demolition, was tested to obtain its dynamic properties. The 10 year old bridge consisted of a continuous flat slab deck of variable thickness having a total width of 103 ft (31.39 m) and spans of 28 ft 6 in. (8.69 m), 71 ft 0 in. (21.64 m), and 42 ft 6 in. (12.95 m). The entire bridge was skewed $10^{\circ}50'$ and the deck was slightly curved in plan.

The mode shapes, natural frequencies, and damping ratios for the lowest five natural modes of vibration were determined using sinusoidal forcing functions from an electrohydraulic shaker. These modes, located at 5.7, 6.4, 8.7, 12.0, and 17.4 Hz, were found to be highly dependent on the lateral properties of the bridge deck. Damping ratios were determined from the widths of resonance peaks. The modal properties from the steady state excitation were compared with those obtained from measurements of traffic-induced vibrations and good agreement was found between the two methods.

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Introduction

The Ottawa River Parkway Service Road Bridge in the City of Ottawa, Ontario, was located just west of the National Library on Wellington Street. Completed in 1967, it carried two lanes of relatively light traffic in each direction; trucks and commercial vehicles were prohibited from using the Parkway. When the Portage Bridge was constructed across the Ottawa River between Ottawa and Hull in 1973, the Service Road Bridge had to be removed to accommodate the approaches for the new bridge. This provided a unique opportunity to test a modern prestressed concrete bridge.

A 2 week period between the date of closure of the bridge and the date of demolition was arranged to permit a series of static and dynamic tests. Unfortunately, the preparation time for the tests was considerably reduced when the demolition date for the bridge was advanced, so that only a limited series of dynamic tests could be performed. This report describes the test program and the results that were obtained.

Description of Bridge

The bridge consisted of a continuous three-span, flat slab deck having a total width of 103 ft (31.39 m) and spans of 28 ft 6 in. (8.69 m), 71 ft 0 in. (21.64 m), and 42 ft 6 in. (12.95 m). The deck was posttensioned in both directions, supported at each end on four sets of V-shaped struts equally spaced across the width of the bridge (Fig. 1). Because the design requirements for the bridge included a shallow centre-span section, both bridge abutments were vertically posttensioned by means of six rock anchors

FIG. 1. View of Ottawa River Parkway Service Road Bridge, looking NW.

at locations shown in Fig. 2. This applied a bending moment to the centre span that countered both dead and live load effects.

The bridge roadway was slightly curved in plan to accommodate a curve in the Parkway. It had a longitudinal gradient of 3% rising towards the east and a transverse gradient of 3.8% rising towards the south. The entire bridge deck, V-supports, and anchorage system were skewed $10^{\circ}50'$ relative to the structural control line of the bridge. The geometry and major dimensions of the bridge are shown in plan in Fig. 2 and in elevation in Fig. 3.

Cables of the posttensioning system of the deck consisted of twelve $\times$ 0.6 in. (15 mm) diameter Freyssinet strands in 3 in. (76 mm) diameter sheaths. The cables were tensioned at both ends to a force varying from 459 to 480 kips (2043 to 2136 kN), depending on location, and were subsequently grouted. The layout of the posttensioning cables is shown in Fig. 4a for a typical longitudinal deck

FIG. 2. Plan view of bridge. Note: all SI lengths given in millimetres.

FIG. 3. South elevation. Note: all SI lengths given in millimetres.

section and in Fig. 4 *b* and *c* for transverse sections over the interior supports. All 52 longitudinal cables had the same profile in elevation, as shown in Fig. 4*a*, and were approximately equally spaced across the deck width. In plan, however, they followed the same curvature as the roadway centreline.

The centre median and the two sidewalks were posttensioned longitudinally (two cables for the median and one cable for each sidewalk) and subsequently bolted to the deck. The transverse cables away from the interior supports were parallel to the top of the bridge deck throughout the width of the bridge.

At the east abutment three vertical rock anchors (see Figs. 2, 3) were tensioned to a cable force of 429 kips (1910 kN) and then grouted within a "tension pile." The same procedure was followed at the west abutment, except that anchorage problems encountered during construction led to replacement of the centre rock anchor by a 40 ft (12.2 m) long

concrete counterweight of 516 kips (2296 kN) centrally hung from the abutment wall.

The base of each of the V-supports at the east end formed a concrete hinge with a reinforced concrete foundation beam, continuous across the full width of the bridge. This beam rested on groups of eight piles under each V-support; as shown in Fig. 3 each pile was inclined at a slope of 1:5. At the west end each of the V-support bases was provided with two André bearings capable of $\pm \frac{1}{2}$ in. (13 mm) transverse movement. These rested on a continuous reinforced concrete foundation beam supported by a group of four piles under each V-support; each pile was inclined at a slope of 1:10 alternating to the east and to the west. All compression piles were of precast reinforced concrete with a bearing capacity of 250 kips (1113 kN) and driven to refusal. The soil consisted of 40 ft (12.2 m) of sandy silt overburden on a shale base.

The specified concrete design strength for the deck

FIG. 4. Layout of posttensioning cables. (a) Longitudinal section (3). (b) Transverse section (1). (c) Transverse section (2).

and V-supports was 5000 psi (34 500 kPa) at 28 days. Tests on two core samples of 2.7 in. (69 mm) diameter and 8.0 in. (203 mm) length, taken from the deck at time of testing, gave ultimate strengths of 6300 and 7000 psi (43 500 and 48 300 kPa) and initial moduli of 4190 and 4630 ksi (28 900 and 32 000 MPa), respectively. The bridge was designed for live loads specified by the American Association of State Highway and Transportation Officials (AASHTO H20-S16).

Testing Program

Traffic Vibrations

Measurement Procedure

Vibration measurements were made on the bridge prior to closure. Traffic across it consisted of passenger cars travelling at approximately 40 mph (64 km/h) and fully loaded tandem and single-axle dump trucks that entered the Parkway from the construction site of the Portage Bridge and crossed the Service Road Bridge at approximately 20 mph (32 km/h).

Willmore Mark II seismometers were used to measure vertical and horizontal vibrations of the bridge deck and V-supports. The locations of seismometers oriented to measure vertical vibrations and placed on both bridge sidewalks are shown in Fig. 5. The other seismometer locations are not shown because their data were not used in subsequent evaluations. The signals from the seismometers were recorded on a 7-channel FM tape recorder. Four 30 min recording sessions were made

FIG. 5. Instrumentation layout on bridge deck. Transducer stations for: ▲ traffic vibrations; □ setup A and ● setup B (both shaker tests).

with various seismometer layouts, but only results from runs 1 and 4 are reported here.

Analysis Procedure

The recorded signals were displayed on a 7-channel Honeywell visicorder. Several paper speeds were used in order to examine the quality of the recorded signals, which were then analysed on a real-time Fourier spectrum analyser. Only the results for stations 3 and 6, located at the midspan of the north and south sidewalks (Fig. 5), were examined in detail.

Forced Harmonic Vibrations

Shaker Description and Operation

Forced harmonic vibrations of the bridge deck were generated by an electrohydraulic inertial shaker with a maximum capacity of 6000 lb (26 700 N)

peak-to-peak force. The shaker system comprises three major components: the power pack, which supplies a 20 gal/min (90 L/min) flow of oil under 3000 psi (20 700 kPa) pressure; the shaker, which consists of a mounting plate and a jack containing the moving piston and attached inertial weights; and a control panel that operates, controls, and monitors the complete shaker system. Figure 6 presents a view of the shaker mounted on the bridge. A number of 100 lb (45.4 kg) inertial weights are attached to the top of the piston. Three servo-valves driven by a sinusoidal voltage from the control panel regulate the flow of oil to the piston, thereby controlling the amplitude and frequency of the piston motion. A sinusoidal vertical motion of the piston would thus impart a sinusoidal vertical force to the supporting structure.

Measurement Procedure

Transducer layout was governed by the requirement for obtaining the lower modes of vibration of the bridge in as few tests as possible, using available transducers and recording channels. Two transducer layouts were chosen for the tests (Fig. 5), designated layouts A and B. At each station an MB Electronics Model 120 velocity transducer having a sensitivity of $98 \text{ mV} \cdot \text{in.}^{-1} \cdot \text{s}$ ($3.86 \text{ V} \cdot \text{m}^{-1} \cdot \text{s}$) was used.

The output signals from the transducers were brought to a mobile laboratory parked near the western approach to the bridge, amplified, and then recorded on a 7-channel FM tape recorder.

Test Procedure

Two shaker locations were selected (Fig. 5) on the bridge deck as points of excitation: No. 1 on the longitudinal centreline of the bridge and No. 2 to one side of the centreline. These locations were chosen so that both symmetrical and unsymmetrical transverse modes of vibration of the bridge could be readily excited.

Ten dynamic tests were carried out. The shaker location, amplitude of the shaker force, frequency range, and transducer layout for each of the ten tests are listed in Table 1. Tests 1 and 2 were preliminary, and were used to check the operation of all instrumentation and to locate, approximately, the natural frequencies of the lower modes of vibration. The results of these tests are therefore not presented. Test 7 was a repetition of test 3, except that it was performed after the backfill at the east abutment had largely been removed. As there were no noticeable differences in the response of the bridge for the two tests, the results of test 7 are also omitted.

For each test the root mean square (rms) output of one of the MB transducers was displayed on a strip chart recorder (1) to select appropriate fre-

FIG. 6. Electrohydraulic shaker mounted on bridge median.

quency increments for the shaker jack so that a well-defined resonance peak could be obtained at each natural frequency of the bridge; (2) to select appropriate frequency increments other than those in the immediate neighbourhood of a natural frequency so that bridge response increments would not be too large; and (3) to ensure that the response of the bridge to a given frequency of excitation had stabilized before the next frequency increment was applied. The latter was generally achieved by holding each frequency constant for 15–20 s.

The amplitude of the shaker force displayed on the control panel was held constant during each test. This required manual adjustment of the gain control to compensate for variations in the force amplitude with frequency.

Analysis of Data

The recorded signals were displayed in the laboratory on a 7-channel Honeywell visicorder. For each test the quality of signals was examined in some detail and the peak amplitudes and phase between signals were determined for each frequency. An example of signals displayed in this manner is shown in Fig. 7. The transition from one frequency to the next is discernible where there is a change in signal amplitude. Initially, an attempt was made to obtain rms amplitudes and phase using a wave analyzer, but this was ineffective and was abandoned.

Results

Traffic Vibrations

Examination of the vibration records showed that in run 1 the peaks of several transducer signals were repeatedly clipped owing to overloading of the tape recorder. This affects the amplitudes of the Fourier spectrum of the clipped signal and will be discussed later.

TABLE 1. Test schedule for forced harmonic vibrations

Test No.	Shaker force		Frequency range* Hz	Shaker location (Fig. 5)	Transducer layout (Fig. 5)	Remarks
	Peak-to-peak (lb)	(N)				
1	1500	6 700	4-15	1	A	Trial runs, results not evaluated
2	2500	11 100	4-15	1	A	
3	4000	17 800	4-15	1	A	
3A	4000	17 800	15-23	1	A	
4A	4000	17 800	4-15-23	1	B	Repetition of test 3 after removal of fill
4B	2000	8 900	4-15-23	1	B	
4C	5500	24 500	4-15-23	1	B	
5	4000	17 800	4-15-23	2	B	
6	4000	17 800	4-15-23	2	A	
7	4000	17 800	4-15	1	A	

*The instrument panel of the shaker required a change in frequency range at 15 Hz.

FIG. 7. Bridge response to shaker excitation, test 6.

The Fourier amplitude spectrum for station 3, run 1, is shown in Fig. 8. Figure 9 *a* and *b* shows the spectra for the addition and subtraction of signals from stations 3 and 6, from which the in-phase or out-of-phase relation of the two stations was determined for each mode of vibration.

The Fourier amplitude spectrum for station 3, run 4, is shown in Fig. 10. The two peaks at 2.7 and 13.5 Hz are surmised to have been caused by construction machinery operating nearby since they are present only in the traffic vibration spectra of run 4. It is also possible that the two peaks represent vehicle suspension resonances. A summary of the natural frequencies derived from the traffic vibration measurements is given in Table 2.

Forced Harmonic Vibrations

Representative results from the forced vibration tests are presented in the frequency-amplitude curves of Figs. 11 and 12. The transducer station and the test number are contained in the caption or labelling of each diagram. The natural modes of vibration of the bridge were determined from these frequency-amplitude curves and from the phase relations of station signals. The natural frequencies are summarized in Table 2 and the associated modes of vibration of the bridge deck are presented in Fig. 13 *a-e*. Alongside each two-dimensional mode shape of the bridge deck are shown the corresponding modal amplitudes along the transverse centreline connecting stations U and L.

FIG. 8. Fourier spectrum of traffic vibrations, run 1, station 3, centre span, north sidewalk.

FIG. 9. (a) Fourier spectrum of sum of signals from stations 3 and 6, run 1. (b) Fourier spectrum of difference of signals from stations 3 and 6, run 1.

Damping Ratio

A modal damping ratio for each natural mode was determined from both the forced vibration data and traffic data, using the half-power bandwidth relation

$$\lambda_i = \Delta f_i / 2f_i$$

where λ_i is the damping ratio; f_i is the natural frequency; Δf_i is the frequency bandwidth at 0.707 times the peak amplitude A_i ; and subscript i refers to the i th mode. For the forced vibration tests A_i is the amplitude at f_i in the frequency-amplitude curve, and for the traffic vibration tests A_i is the Fourier spectrum amplitude at f_i .

As the widths of resonant peaks are affected when

FIG. 10. Fourier spectrum of traffic vibrations, run 4, station 3, centre span, north sidewalk.

two modes are very close, the value of the modal damping ratio computed for either or both of the closely spaced modes may be somewhat less reliable than that of well-spaced modes. This is the case for mode 2, although not for mode 1, in the frequency-amplitude curves of the forced vibration tests. Because signals from a symmetrically placed transducer station such as station K do not contain significant contributions of antisymmetric modes, the resonant peak for mode 1 (Fig. 11b), a symmetric mode, is virtually free of interference from mode 2, an antisymmetric mode. Unfortunately, no data were available from the forced vibration tests that would have eliminated this effect for mode 2. If a symmetrical placement of transducers relative to the longitudinal bridge centreline had been employed with shaker location 2, resonant peaks of all symmetrical modes could have been eliminated by subtracting the signals from the symmetrically placed transducers, leaving only the resonant peaks of the antisymmetric modes. This was demonstrated for the traffic vibration spectra in Fig. 9 a and b.

Damping ratios for run 1 (traffic vibrations) were calculated from Fig. 9 a and b, which contains the spectra obtained from the addition and subtraction of the signals from stations 3 and 6. Both spectra have almost no modal interference; stations 3 and 6 are located on the transverse centreline on opposite sides of the bridge. As station 6 was not recorded, it was not possible to employ a similar technique for the data from run 4. Thus, the damping ratios of modes 1 and 2, obtained from the Fourier amplitude spectrum of station 3 (Fig. 10) may have been affected by the interference between these modes.

Although the damping ratios from run 1 were not influenced by modal interference, they may have been affected by signal clipping that occurred during tape recording. A clipped signal results in a smaller Fourier amplitude at frequencies that contribute significantly to the amplitude of the signal. As the ratio is calculated from the width of the spectrum peak at 0.707 of peak amplitude, a lowering of the peak will result in a slightly higher damping ratio

TABLE 2. Modal properties as determined from forced harmonic vibration tests and vibrations induced by traffic

Mode	Forced harmonic vibrations		Traffic vibrations		
	Frequency (Hz)	Damping ratio (% critical)	Frequency (Hz)	Damping ratio (% critical)	
				Run 1	Run 4
1	5.7	2.2, 2.2	5.7	2.6	2.1
2	6.3	2.6	6.3	2.4	2.3
3	6.4	1.9	8.6	1.7	1.4
	8.7	1.5			
4	12.0	1.6	8.7	2.1	2.2
		2.1	11.8		
5	17.4	1.8	12.0	2.4	1.7
		1.9	16.8		

than would otherwise have been obtained. It is judged, however, that for run 1 the calculated damping values are overestimated by not more than about 10%. The modal damping ratios calculated from runs 1 and 4 are given in Table 2.

For symmetric modes 1, 3, and 5 (forced harmonic vibrations) only amplitude-frequency curves for station K and shaker location 1 were used to calculate the modal damping ratios. These were the curves judged to be essentially free of interference from adjacent modes (see Fig. 11b). Table 2 contains a summary of the modal damping ratios determined from the forced vibration tests.

Discussion of Results

In the forced vibration tests two shaker locations, one on the longitudinal centreline of the bridge and the other to one side of the centreline, were chosen so that all significant symmetrical and antisymmetrical transverse modes of vibration of the bridge would be readily excited. The necessity of selecting two such locations of excitation can be seen from the amplitude-frequency curves. With the shaker at location 1 the amplitude-frequency curves in Fig. 11 *a* and *b* show an almost complete absence of the transverse anti-symmetrical modes at 6.3 and 12.0 Hz. As location 1 was on the longitudinal centreline of the bridge, the two antisymmetrical modes were barely detectable (Fig. 13 *b*, *d*). When the shaker was moved to location 2, to one side of the longitudinal centreline, the two transverse antisymmetrical modes were easily excited (Fig. 12 *a*, *b*).

This consideration does not arise with traffic-induced vibrations because it is more than likely that all significant modes of vibration are excited by the usual mix of vehicle type, speed, driving path, direction of travel, and impact of vehicles on entering and leaving the bridge. For both the forced vibration

FIG. 11. (a) Frequency response curve for station U, test 4B. (b) Frequency response curve for station K, test 4C.

and traffic vibration tests, however, it is still possible that a significant natural frequency might go undetected, even when excited, if too few transducer stations were used and if they all happened to be placed at or near modal nodes.

The Fourier amplitude spectra derived from the traffic vibrations and the amplitude-frequency curves

FIG. 12. (a) Frequency response curve for station X, test 5. (b) Frequency response curve for station AA, test 5.

for the forced vibration tests for shaker location 2 are qualitatively similar throughout the given frequency range. This similarity is especially striking when results for comparable transducer stations are examined, (for example, station 3, Figs. 8, 10; station AA, Fig. 12b).

The mode shapes and modal frequencies determined from the forced vibration tests demonstrate the two-dimensional nature of the bridge deck. Except for mode 5, the mode shapes of the deck in the longitudinal direction are similar to the fundamental mode shape of a three-span beam element; in the transverse direction the deck has a mode shape that contains the number of nodes corresponding to the mode number. The two-dimensional nature of the bridge deck is also demonstrated by the sequence of modal frequencies, since they are more closely spaced than would be the case for one-dimensional deck behaviour. Because of this close spacing, the contribution of modes other than the fundamental should also be considered when the response of the bridge deck to dynamic loads from traffic, earthquakes, etc., is calculated.

The summary of data in Table 2 shows that the

FIG. 13. Mode shapes of bridge deck. (a) Mode 1: 5.7 Hz. (b) Mode 2: 6.4 Hz. (c) Mode 3: 8.7 Hz. (d) Mode 4: 12.0 Hz. (e) Mode 5: 17.4 Hz.

natural frequencies of the lower modes of vibration for both the traffic and forced vibration tests are virtually the same. For higher modes, however, the natural frequencies from the traffic vibration tests

are slightly lower than those from the forced vibration tests. This could be attributed to the slight increase in the modal masses of the bridge as a result of the vehicle loads. As the equivalent modal mass for the higher modes is smaller than that for the lower ones, the additional mass from travelling vehicles would have a greater effect on the higher modal frequencies than on the lower ones. For this bridge the differences in modal frequencies for the two test procedures were not that significant. For other bridges this could change, however, depending on the size of the bridge and the type of traffic moving across it. A preponderance of heavy vehicles across this bridge might also have produced some differences in the natural frequencies of the lower modes.

The damping ratios derived from the traffic and forced vibration tests, although not identical, are quite similar in magnitude. Damping ratios from run 1 of the traffic vibrations tend to be somewhat larger than those from the forced vibration tests. This may, in part, be a result of signal clipping that occurred during the recording stage.

The modal amplitudes, as determined from both the traffic and forced vibration tests, indicate that the south side of the bridge has larger centre-span modal amplitudes than the north side. A similar trend can be observed in the side-span modal amplitudes at stations M and N. This difference is most pronounced in mode 1, Fig. 13. It appears from these results that the north side of the bridge deck has stiffness properties slightly different from those of the south side. This could be due to differences in the posttensioning system or to differences in abutment anchorages of the bridge.

Conclusions

Dynamic tests conducted on the Ottawa River Parkway Service Road Bridge demonstrate the following.

1. The dynamic properties (natural frequencies, mode shapes, and modal damping ratios) of this type of bridge structure can readily be determined by forced harmonic vibration tests using a shaker that generates a sinusoidal force with controlled frequency and amplitude. Care needs to be exercised in the choice of shaker location and transducer layout if one wishes to obtain a complete set of natural modes of vibration.

2. The natural modes of vibration identified for this bridge have mode shapes that depend, to a large

extent, on the transverse properties of the bridge. This results in a number of modes of vibration having natural frequencies close to that of the fundamental. Because of this close spacing of modes, it is necessary to consider the contribution of these lower modes, in addition to the fundamental, when the response of the bridge to traffic or to other types of dynamic loading is determined.

3. Natural frequencies obtained from traffic vibration tests agree closely with those from the forced harmonic vibration tests. Mode shape comparisons were not possible because traffic measurements were not conducted in sufficient detail. There is every indication, however, that a full set of dynamic properties could also be obtained from traffic vibration tests. As these are simpler to perform than forced harmonic vibration tests, they could be used to advantage in many practical situations in determining the dynamic properties of similar types of bridges. The effect of vehicular mass on the dynamic properties of the bridge and the presence of spurious frequency peaks in the Fourier amplitude spectra from sources such as nearby construction machinery, traffic, or instrumentation characteristics need to be considered in interpreting the results.

4. Modal damping ratios derived from the forced vibration tests and traffic vibration tests are quite similar in magnitude.

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