Dynamic Load Testing of Highway Bridges

RETO CANTIENI

ABSTRACT

Between 1958 and 1981 the Swiss Federal Laboratories for Materials Testing and Research (EMPA) performed dynamic load tests on 226 beam and slab-type highway bridges; most of them were concrete structures. Test procedures as well as measurement and data processing techniques are briefly described. As a certain degree of standardization was maintained over the years, it was possible to draw some general conclusions from the summarized test results. In particular, the relationship between fundamental frequency and maximum span is discussed as well as the measured damping values. Additionally it is shown that a highway bridge exhibits a pronounced dynamic response only if (a) its fundamental frequency lies in the same region as one of the two vehicle modes of concern and (b) the vehicle speed and pavement roughness are tuned to each other so that the corresponding vibrations of the vehicle will be excited. As a consequence, it has been proposed to define the dynamic load allowance for highway bridge traffic loads as a function of the bridge's fundamental frequency in the new Swiss Code. Taking advantage of the rapid development in the field of electronics, new data acquisition and processing methods have been introduced at EMPA in the last few years. The synchronous measurement and recording of the test vehicle's dynamic wheel loads and the bridge response is now possible as well as subsequent digital signal analysis.

Load tests on bridges have a long tradition at the Swiss Federal Laboratories for Materials Testing and Research (EMPA). The first publications on such tests still accessible today stem from the work of M. Roš, who held a leading position at EMPA from 1924 until 1949. He described the static and dynamic load tests performed in 1922 on the Kettenbrücke (iron chain suspension bridge) in Aarau (1). His legendary EMPA Report 99 (2) gives the procedures and results of load tests undertaken between 1923 and 1947 on 32 arch and 14 beam-type bridges, including the famous structures of R. Maillart. Nearly all of these structures, most of them highway bridges, were subjected to static as well as dynamic loads.

In 1960 Rösli published some results of dynamic tests on 20 prestressed concrete highway bridges that were performed by EMPA in the years following the Roš era (3). Some of the test reports from that period are still available. They contain the data that were used to form the basis for summarizing results of dynamic tests on 226 highway bridges performed by EMPA since 1958 and presented later.

The question can be asked, how has it been possible, in a small country like Switzerland, to gather a considerable amount of experience in the field of dynamic load testing of highway bridges. A comparison of the various code provisions in force since 1892 shows that dynamic tests were required only between 1892 and 1913 and again since 1970. Therefore, it has been largely due to the energies and competence of the EMPA staff that the majority of static tests on highway bridges, which have been required in Switzerland since 1892, were supplemented by dynamic tests. It has not always been a simple task to convince the respective clients (mainly Cantonal highway administrations) of the advantages of dynamic tests.

There has usually been no reason to question the applicability of the results of static tests. Such tests, for example, allow comparisons of the measured behavior of the structure with the respective calculations. The situation has been somewhat different for dynamic tests. The results of these tests, such as natural frequencies, damping, and dynamic increments (also referred to as impact factor, dynamic load factor, or dynamic coefficient) could neither be compared with calculated values nor with corresponding data from code provisions. Even though the state-of-the-art has advanced considerably during recent years, this statement can be revised only slightly because

- Computing the frequencies of a sufficient number of bridge natural modes is no longer a problem today.
- The great number of attempts undertaken in the last decades to predict damping values or dynamic increments on a theoretical basis have not led, as far as is known to the author, to any generally adopted and easily applicable solutions.
- The qualitative and quantitative comparison of measured dynamic increments with corresponding code provisions is generally still not possible. [The 1979 Ontario Highway Bridge Design Code is an exception (4).]

On the one hand, this is mainly because the loads (to which the dynamic increments are related) used in the load tests are not the same as those in the codes. On the other hand, the question can be asked whether the dynamic effects of moving trucks on the response of a bridge are really predominantly a function of the average bridge span, as assumed, for example, in the present Swiss Code (5).

The dynamic increment, as used in the design of a bridge, is the value of greatest practical interest resulting from dynamic load tests. If straightforward comparison of the measured values with theoretical analyses or code provisions is not feasible, there remains the possibility of comparing the experimental values among each other. This can obviously only be done in a significant way if the procedures of testing, measuring, and data processing are standardized as much as possible. Being aware of this fact, the decision was made by Rösli and Voellmy in the 1950s to standardize the EMPA dynamic load tests on highway bridges. Although EMPA is not the only institution performing such tests in Switzerland, the number of standard tests performed by EMPA since that time is considerable.

Only in the last several years has the client been able to profit from this experience. Now the results of a current load test are related to the results of all previously performed tests. Additionally interpretation of the collected test data has

FIGURE 1 A two-axle vehicle before passing over a contact threshold (speed measurement) and over a 45 mm (1.8 in.) thick plank.

led to a proposal for the definition of the dynamic increment for a future Swiss Loading Code.

STANDARD DYNAMIC LOAD TESTS AS PERFORMED SINCE 1958

Test Procedures

The bridges are tested dynamically through passages of a single, fully loaded, two-axle truck, which is provided by the client. With a normal axle spacing of 4.5 m (15 ft) the gross weight of the vehicle usually lies near the legal limit of 160 kN (35 kips). (In general 280 kN or 62 kips is the maximum legal gross weight limit in Switzerland for any type of vehicle.)

The test vehicle is driven at constant speed, whenever possible, along the longitudinal bridge axis and always in the same direction (Figure 1). The tests are begun with a vehicle speed of 5 km/h (3 mph) which is then increased after every passage in steps of 5 to 10 km/h (3 to 6 mph) up to the maximum achievable speed. Tests on the undisturbed pavement are repeated with a plank placed on the roadway in the main measurement cross section. The plank is approximately 50 mm (2 in.) thick and 300 mm (12 in.) wide.

Data Acquisition

Deflection is measured whenever possible at the characteristic point of the bridge, which is normally at the midpoint of the maximum span (main measurement cross section). In many instances deflection is measured at additional points of the superstructure. This measurement is provided by an invar wire stretched between the measurement point at the structure and a fixed reference point under the bridge. In about 1964 the mechanical vibration recorders (vibrographs) that registered the deflection signal on a rotating cylinder were replaced by electronic measurement setups. These usually consist of an inductive displacement transducer, a signal amplifier, and an ink recorder.

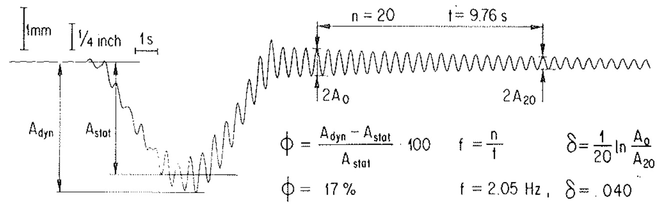


FIGURE 2 Deflection of the midpoint of the Ponte di Campagna Nova, which has one 45-m (148-ft) span, under the passage of a 160 kN (35 kip) vehicle traveling at 25 km/h (16 mph) on the undisturbed pavement (f = fundamental frequency, δ = logarithmic decrement, and ϕ = dynamic increment).

Data Processing

From the dynamic deflection signals, registered on paper strips, the following information can usually be obtained (Figure 2):

1. The frequency of one or more modes of the bridge,
2. Damping of the natural vibration dominant in free decay, and
3. The dynamic increment of one or more measurement signals as a function of the vehicle speed.

Natural frequency and associated damping can be determined by manual signal analysis in the time domain only if the bridge vibrations decay harmonically after the passage of the vehicle. If an accurate time measure has been recorded along with the signal, the natural frequency, Hz, can be established by counting the number of periods in a given portion of the decay process:

$$f = n/t$$

where

n = number of periods, and
t = corresponding time interval in seconds.

Damping, for example the logarithmic decrement δ, can be determined from the same time interval. This requires measurement of the magnitude of the first and last amplitudes having the same phase

$$\delta = 1/n\ (\ln)(A_0/A_n)$$

where

n = number of periods (n+1 amplitudes), and
ln = natural logarithm.

The percentage of critical damping, p, is given by

$$p = (\delta/2\pi) \cdot 100.$$

The dynamic increment is defined as

$$\phi = (A_{dyn} - A_{stat})/A_{stat} \cdot 100$$

where A_{dyn} is the peak value of the bridge response during a passage of the test vehicle and A_{stat} is the peak value of deflection observed under static loading with the same vehicle. Although the readout of A_{dyn} from a paper strip can be performed in a straightforward manner, the determination of A_{stat} may be problematic.

Two basically different methods have been used. In earlier years, A_{stat} was estimated for every passage from the same dynamic trace as the corresponding A_{dyn}. With increasing dynamic response of a bridge, the quality of this estimate for A_{stat} decreases. Therefore, in recent years A_{stat} has been determined from the traces of three very slow passages (crawl tests), and the value thus obtained is subsequently used in the evaluation of all the passages. Comparative tests in 1976 showed that both methods yield the same value for A_{stat} if the driving lane is marked and accurately maintained by the driver.

There is an additional problem in determining the dynamic increment, ϕ. Aside from all the known quantities such as vehicle properties and speed, pavement roughness, and bridge properties, the position of the measurement point in the cross section of the bridge was found, under certain circumstances, to exert a significant influence on the calculated value of the dynamic increment.

This is because the deflection distribution over a bridge cross section under static and dynamic loading is generally not identical. Experience shows that on the one hand, the deflection (A_{stat}) of a statically loaded beam-type bridge will always vary more or less strongly over its cross section, depending on the shape and stiffness of the section. On the other hand, the vertical motional amplitudes (responsible for A_{dyn}) of the same bridge under dynamic loading will remain constant over the cross section, as long as the bridge response consists solely of longitudinal bending modes.

The dynamic increment, ϕ, as a relation between A_{dyn} and A_{stat} therefore will depend on the exact location of the measurement point in the bridge cross section. This dependency is almost negligible for bridges with stiff box-shaped cross sections but may invalidate results gathered from bridges with wide and flexible I-beam cross sections.

RESULTS OF TESTS ON 226 HIGHWAY BRIDGES

In the years from 1958 to 1981 the concrete structures section of EMPA performed load tests on 356 bridges. In the present context, the standardized dynamic part of 226 combined tests on beam and slab-type highway bridges is of interest. The remaining 130 tests concerned tests on other bridge types and purely static tests. Of the 226 bridges, 205 are prestressed concrete, 5 reinforced concrete, 14 composite steel and concrete, and 2 prestressed lightweight concrete structures. The number of spans varies between 1 and 42 with an average of 4 and a value of peak occurrence of 3. The most common structural systems are the continuous beam over more than one span (72 percent) and the simply supported one-span beam (12 percent). Total lengths and lengths of the maximum span can be characterized as follows:

	Total Length	Maximum Span
Minimum	13.0 m (43 ft)	11.0 m (36 ft)
Maximum	3,147.5 m (10,300 ft)	118.8 m (390 ft)
Mean	155.9 m (511 ft)	39.5 m (130 ft)

Of the bridges tested, 109 are straight and without skew, 97 are skewed or curved, and 20 are both skewed and curved. Half of the bridges have a box-shaped cross section, 26 percent a multibeam deck, and 24 percent a solid or hollow-core slab cross section. The cross-sectional width varies between 4.3 m (14 ft) and 30.4 m (100 ft) with a mean value of 12.9 m (42 ft).

The spring constant, defined as gross weight of the test vehicle divided by A_{stat}, was found to lie between 7 kN/mm (40 kip/in.) and 800 kN/mm (4,500 kip/in.) with a mean value of 173 kN/mm (1,000 kip/in.) and a value of maximum occurrence of 75 kN/mm (430 kip/in.).

These data are taken from a computer data bank set up in 1981 in which up to 40 parameters from each of the 226 dynamic load tests are stored. Detailed information on the distributions of these parameters (test conditions as well as results) is given in EMPA Report 211 (6). The most important findings derived from these data are presented in the paragraphs that follow. The number of measured values displayed in the different figures does not equal 226 because (a) it was not possible to meet all the requirements of a standard test as described previously for all tests and (b) the value of concern could not always be determined from the recorded signals.

Fundamental Frequencies

Figure 3 shows the distribution of fundamental frequencies measured on 224 bridges. The result of an attempt to establish a relation between the fundamental frequency of a bridge and the length of its maximum span is shown in Figure 4. The scatter of

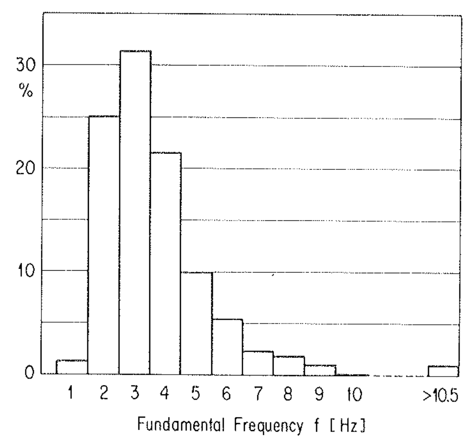


FIGURE 3 Distribution of the fundamental frequencies, f, measured on 224 bridges.

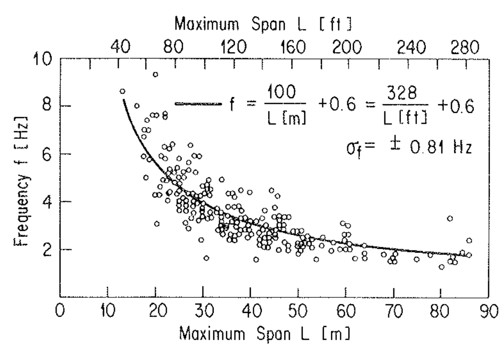


FIGURE 4 Fundamental frequency, f, as a function of the maximum span, L, for 224 bridges (σ_f = standard deviation).

the measured values around a curve determined through nonlinear regression is considerable. This is not surprising in view of the large variations in geometry and stiffness of the bridges tested. To achieve a smaller scatter, the following limitations were introduced:

- Eliminate cantilevered structures,

- Limit the horizontal radius of curvature of the longitudinal bridge axis to ≥900 m (3,000 ft),
- Limit the skew to ≤15 deg.,
- Limit the spring constant, k, to 70 kN/mm ≤ k ≤ 270 kN/mm (400 kip/in. ≤ k ≤ 1,540 kip/in.),
- Eliminate results that were not obtained from measurements on the maximum span.

The regression function calculated from the remaining 100 values is almost identical to the function shown in Figure 4 but the scatter has diminished from $\sigma_f = \pm 0.81$ Hz to $\sigma_f = \pm 0.62$ Hz. As will be shown later, the dynamic increment will probably be defined in a future Swiss Code on bridge loading as a function of the bridge's fundamental frequency. Therefore, the previously mentioned equation to estimate this frequency from the maximum bridge span may be of some practical value in the early design stages.

Damping

The distribution of the logarithmic decrement, δ, determined from the free decay process of 198 concrete bridges is shown in Figure 5. Because the δ-values are scattered over a considerable range, an attempt was made to separate the bridges with relatively strong damping from those with relatively weak damping.

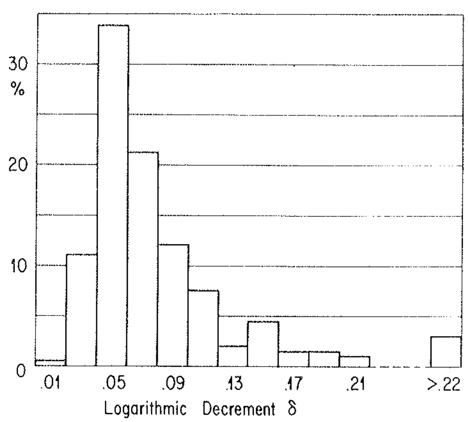


FIGURE 5 Distribution of the logarithmic decrement, δ, measured on 198 concrete bridges (minimum, 0.019; mean, 0.082; maximum, 0.360).

Considering the superstructure of a bridge as a vibrating body, overall damping can be separated into internal or structural damping and external or system damping. Thus, structural damping is due to energy dissipation during all kinds of vibrations of the superstructure; and system damping is due to energy dissipation during relative movements between superstructures and substructures and during all kinds of vibrations of the substructure elements. Because only concrete bridges were taken into account, material damping was not considered. Information was not available on the influence of vibrational amplitude on damping; therefore, the damping relation between this amplitude and the maximum span could not be taken into account. As will be seen later, this influence does not seem to be significant when compared with the factors actually under consideration.

Concerning structural damping it was found that bridges responding predominantly in a longitudinal mode of flexure have on the average a smaller logarithmic decrement than bridges that respond in a superposition of modes of longitudinal flexure, torsion, and transverse flexure. Straight structures with narrow, closed cross sections showed a mean $\delta = 0.063$ (p = 1 percent), and curved or skewed bridges with wide cross sections showed a mean $\delta = 0.087$ (p = 1.38 percent).

The EMPA data bank does not contain the necessary information to investigate system damping in detail. For example the type of bearing constructions and the relative stiffness of piers are often not given in the test reports. An analysis to confirm the results of Green (7) showed that the total length of a bridge indicates the damping to be expected. Long bridges with a total length of more than 125 m (410 ft) showed a mean $\delta = 0.056$ (p = 0.89 percent), and short bridges with a total length of less than 75 m (246 ft) a mean $\delta = 0.098$ (p = 1.56 percent). Although the total length of a bridge surely influences the previously mentioned parameters of structural damping, it presumably also reflects influences of system damping. The long structures have an average of 0.26 supports per 100 m (33 ft) of bridge length, the short ones a corresponding value of 0.60.

Figure 6 shows the δ-distributions for two classes of bridges, which were formed by combining the parameters of structural damping and the total length: (a) long, straight bridges with narrow, closed cross sections and (b) short, curved and/or skewed bridges with wide cross sections. These two classes seem to have well separated mean values and distributions of overall damping.

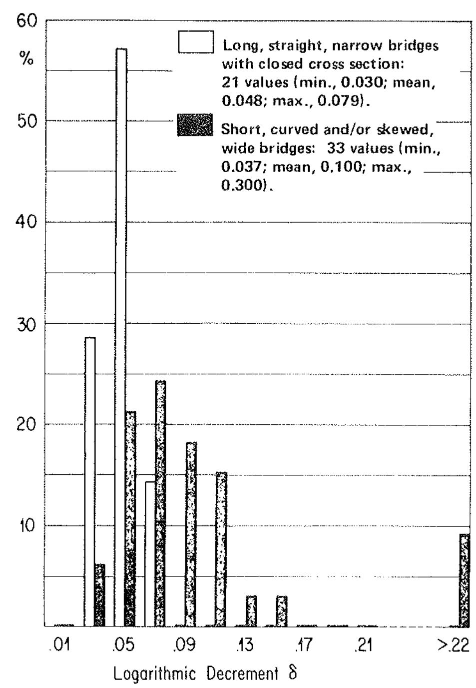


FIGURE 6 Distributions of the δ-values for two classes of concrete bridges.

Dynamic Increments

Where a falsifying influence of the location of the measurement point in the body cross section was to be expected, these values were eliminated before trying to interpret the measured dynamic increments, ϕ. When the measurement point lies outside the region of direct influence of the test vehicle, the resulting ϕ will be too high to a lesser or greater

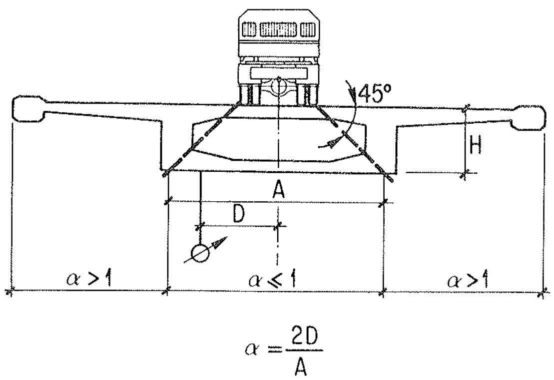


FIGURE 7 Definition of the coefficient, α, describing the location of the measurement point in the bridge cross section relative to the direct influence region of the vehicle.

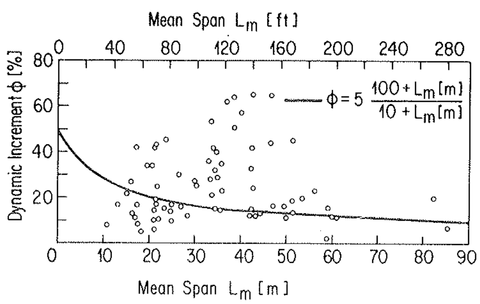


FIGURE 8 Dynamic increments, ϕ, for passages without a plank as a function of the mean span L_m for 73 concrete bridges ($\alpha \leq 0.8$). The solid line indicates the provision in the present Swiss Code (5).

extent depending on the transverse bending stiffness of the bridge. Therefore, only data corresponding to $\alpha \leq 0.8$ (see Figure 7 for the definition of α) were retained.

First the relationship between the dynamic increment and the mean span of the bridge was investigated. It can easily be seen from Figure 8 that this relationship does not correspond to the function implied from the present Swiss Code (5). Each of the measured ϕ-values displayed in Figure 8 represents the peak value of the dynamic increments established from a standard test with the vehicle passing over the undisturbed pavement. As mentioned earlier the comparison between measured values and code provisions can be qualitative only.

Then it was found that the fundamental frequency is an important parameter influencing the response of a bridge to the passage of a test vehicle. A general explanation for this observation is as follows. The bridge as well as the vehicle are mechanical mass/spring/damper systems whose dynamic behavior is determined by natural modes; the bridge response will therefore be influenced by the relationship between the frequencies of these modes. Stated differently, one of the conditions to be fulfilled for a distinct bridge response is a matching of the frequencies of dynamic wheel loads and the natural mode of a bridge.

The natural modes of the EMPA test vehicles were presumably scattered over a relatively narrow frequency band. The vehicles were all fully loaded two-axle tip trucks with leaf springs. Under normal conditions of pavement roughness, the dynamic wheel loads of such vehicles occur mainly in two frequency ranges: (a) in the range of the body bounce frequency between ~2 Hz and ~5 Hz and (b) in the range of the wheel hop frequency between ~10 Hz and ~15 Hz. The body bounce mode of a vehicle is excited by relatively long waves, and the wheel hop mode by relatively short waves of the roadway unevenness. Depending on the vehicle speed, an unevenness of a certain length may be effective in both ranges. As an example of special interest, a plank excites body bounce vibrations for low vehicle speeds and almost pure wheel hop vibrations for speeds above 40 km/h (25 mph).

Figure 9 shows the maximum dynamic increments, ϕ, of test series on the undisturbed pavement as a function of the bridge's fundamental frequency. The curve encompassing the measured values shows a clear peak in the region of 2.5 to 4 Hz (i.e., in the region of the vehicle's body bounce frequency). The statement that the shortwave amplitudes of normal pavements are too small to excite the vehicle's wheel hop mode significantly is based on one single measurement value in the corresponding frequency range.

From the equivalent diagram for the test series with a plank lying on the roadway (Figure 10) it can be seen that a first resonance peak lies in the range of 1.5 to 3 Hz and a second peak at frequencies above 7 Hz. Taking into account that (a) a plank represents a large amplitude of excitation compared with the amplitudes of roughness of a usual pavement and that (b) the natural frequencies of a nonlinear system, such as a leaf-sprung truck, decrease with increasing amplitude of excitation, then the two observed peaks lie in the range of the body bounce and the wheel hop frequencies of the vehicles, respectively.

It can also be seen from Figures 9 and 10 that the dynamic increments do not follow a one- or two-peaked resonance curve. Instead they are scattered below the peaks. Unfortunately the information available is not sufficient to determine the reasons

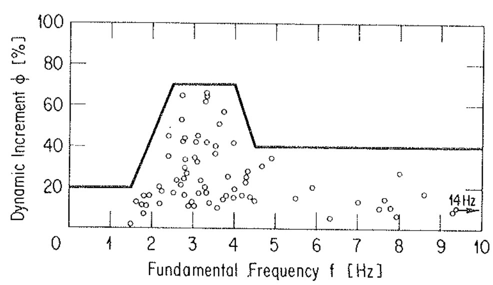


FIGURE 9 Dynamic increments, ϕ, for passages without a plank as a function of the fundamental frequency, f, for 73 concrete bridges ($\alpha \leq 0.8$).

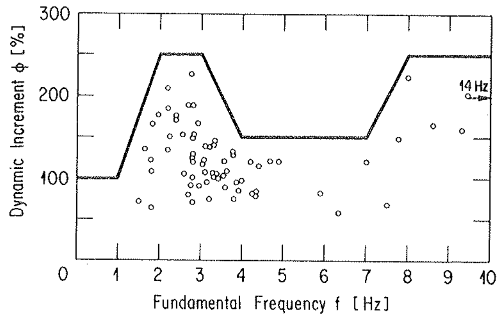


FIGURE 10 Dynamic increments, ϕ, for passages with a plank as a function of the fundamental frequency, f, for 69 concrete bridges ($\alpha \leq 0.8$).

for this scatter in detail. Nevertheless, an attempt was made to evaluate the influence of three parameters available in the EMPA data bank (6). It was found that damping does not appear to exert an important influence on the dynamic increment and that straight, beam-type bridges respond only slightly more strongly than more complicated structures. The investigation also showed that the influence of pavement roughness on dynamic increments for passages without a plank was greater for those bridges with medium or poor pavements than for those with smooth pavements.

Hence, the interpretation of the EMPA test results can be summarized as follows: A highway bridge exhibits a pronounced dynamic response only if (a) its fundamental frequency lies in the same region as one of the two vehicle modes of concern and (b) the vehicle speed and pavement roughness are tuned to each other so the corresponding vibrations of the vehicle will be excited.

The solid curves indicated in Figures 9 and 10 were proposed to be integrated qualitatively into the new version of the Swiss Code on Highway Bridge Loadings. The curve shown in Figure 9 is similar to the curve used in the 1979 Ontario Highway Bridge Design Code (4). However, these two curves were developed independently and were based on two different sets of experimental data.

CURRENT EMPA TESTING METHODS

Scheduled Tests with a Single Vehicle

To reduce variations in the load parameters further, several improvements have been introduced in recent years. Unfortunately EMPA has not been able to obtain its own test vehicle. It is now possible, however, to make use of the same Army vehicle for all the tests. Payload, tires, and tire pressure are always the same, so it can be assumed that the dynamic properties of the vehicle remain approximately constant.

In addition the driving lane is marked with rubber cones or paint (Figure 1) and the vehicle speed is controlled with a special test wheel (Figure 11). Thus it is possible to maintain constant speed

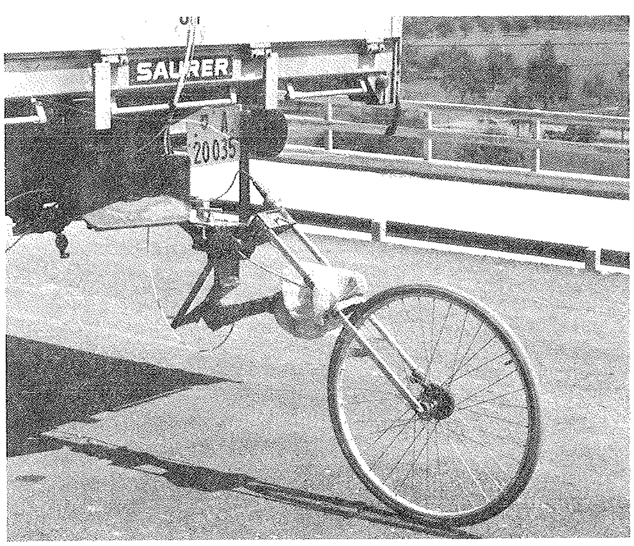


FIGURE 11 With the help of a test wheel, the speed can be accurately measured and controlled during the entire passage.

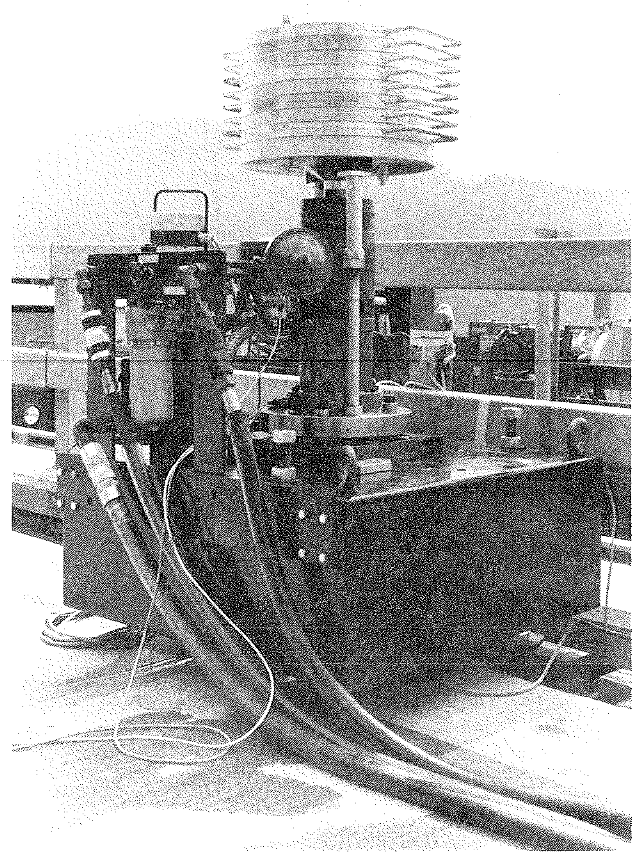


FIGURE 12 Servohydraulic vibration generator (SCHENCK).

within ±0.5 km/h (0.3 mph) from 2 km/h (1.2 mph) up to 100 km/h (62 mph). Furthermore, additional passages are carried out in the range of critical speed, where the bridge response reaches its peak value. In this range, detected by on-line preprocessing of the measurement signals, the speed increment, Δv, is thus reduced to 1 to 3 km/h (0.6 to 1.9 mph).

Excitation with a Servohydraulic Vibration Generator

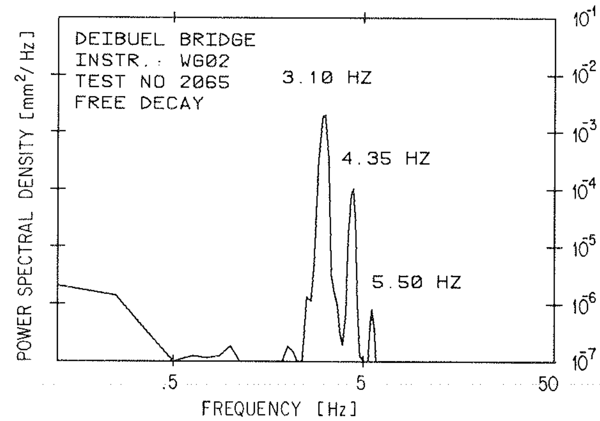
The natural frequencies of a bridge can be determined very precisely through swept-sine excitation with a vibration generator. In 1977 EMPA purchased the servohydraulic actuator shown in Figure 12. The whole system is mobile and produces a sinusoidal force with a peak value of 5 kN (1.1 kip) in the frequency range between 2.3 Hz and 20 Hz. For frequencies below 2.3 Hz the force decreases with the square of the frequency because the piston stroke is limited to ± 50 mm (± 2 in.). The most important advantage of such a vibration generator is the high frequency resolution that can be achieved. If a suitable function generator is used to produce the drive signal, this resolution may be as high as 0.001 Hz. After looking for the natural frequencies first in a relatively quick sweep, these can later be evaluated in greater detail.

Impulse-Type Excitation

Determination of the natural frequencies or in fact all modal parameters of a structure through impulse-type excitation is common in the investigation of machines, for example, tool machines. The structure to be analyzed is struck with single hammer blows and the input force and system response (acceleration) are measured to yield the information necessary for the modal analysis. In contrast to a swept-sine excitation as described in the previous paragraph with a long duration input time signal and a very narrow-band frequency spectrum, an impulse-type excitation provides a time signal of very short duration with a correspondingly broad frequency spectrum.

The advantages of the impulse-type excitation are obvious: (a) exciting with a portable hammer is much less expensive than exciting with a complete servo-hydraulic system, and (b) one hammer blow excites all modes simultaneously whereas sweeping through the frequency band of concern may be time consuming.

Disadvantages have to be considered when applying impulse-type excitation to highway bridges. Due to relatively strong damping, the amplitudes of the bridge response may not be significant for a time long enough to allow the determination of a spectrum with a reasonable frequency resolution. This frequency resolution (i.e., the distance between two lines in the spectrum) equals one time window (length of the signal in seconds). Considering a mean logarithmic decrement for bridges of 0.082 (p = 1.3 percent, see Figure 5), and a corresponding fundamental frequency of 3 Hz, the initial amplitude will be damped out to 50 percent after 2.8 sec and to 1 percent after 18.7 sec. Thus, assuming an initial signal-to-noise ratio of at least 40 dB for such a free decay process and transforming one 20 sec time window, a frequency resolution of 0.05 Hz can be achieved. This resolution is considerably lower than that resulting from a swept-sine excitation test.



Note: WG 02: Deflection at the midpoint of the largest span. Horizontal axis: Frequency 0-50 Hz, 400 lines, logarithmic scale. Vertical axis: Mean-squared displacement density, full scale = 0.1 mm^2/Hz (divide by 645 to convert to sq. in./Hz), logarithmic scale.

FIGURE 13 Power spectral density (PSD) obtained from the free decay of a three-span prestressed concrete bridge, 32 m, 41 m, and 36 m (105 ft, 135 ft, and 120 ft). The frequencies of the first three modes are indicated.

CURRENT DATA ACQUISITION AND PROCESSING METHODS

Digital Signal Analysis

With measurement signals recorded on paper strips, data processing is possible in the time domain only and has to be performed manually with the help of pencil and ruler. Several problems occurring with this kind of data processing have been mentioned earlier. To solve these problems and above all to allow digital data processing and signal analysis in the frequency domain, a Pulse-Code-Modulation (PCM) system was purchased by EMPA in 1977. Analog signals from up to 32 transducer/amplifier units are digitized by this system with 12-bit resolution and then recorded in digital form with a tape unit. Upon playback, the signals can be transferred directly in digital form to the disk of a computer. With the help of corresponding software, standard time domain analysis can then be performed simultaneously for all signals.

Frequencies of the bridge's natural modes are determined by transforming the time signals into frequencies (i.e., by calculating power spectra, Figure 13). This task is performed by a Fast Fourier Transform (FFT) analyzer, Nicolet 660A, directly linked with the computer. Because the FFT analyzer is able to treat two time signals simultaneously, it is possible to investigate the phase and amplitude relationships between two signals as a function of frequency. Hence, the shapes of all modes contributing to the signals can be determined.

Measurement of Dynamic Wheel Loads

Until sufficient knowledge of the test vehicle's dynamic wheel loads is acquired, no detailed interpretation of the bridge response will be possible. The first successful attempts to solve the problem of continuously measuring dynamic forces between tires and the riding surface were made in the United States and Germany (8,9).

The dynamic wheel loads of a highway vehicle can be measured directly or indirectly. With the direct method the normal wheel hubs are replaced by specially designed and instrumented measurement hubs (force transducers). This method is very accurate but also very expensive (10). With the indirect method, the tire is used as a measurement spring (i.e., the deformation of the tire is measured instead of the force). The necessary calibration curve of the force versus deformation relationship must then be determined in an additional static test. Comparative tests showed that vertical tire deflection reflects the actual wheel load with sufficient accuracy (11).

Therefore, in 1977 EMPA acquired an opto-electronic system that measures the distance between the vehicle's axle and the riding surface (i.e., the vertical tire deformation) with the aid of an infrared beam reflected on the pavement (Figure 14). The main parts of the measurement setup are an infrared emitter, a reception camera, and an electronic control unit. The vertically oriented infrared emitters are located next to the tires; the receivers are slanted at approximately 45 degrees. The infrared light with a wavelength of 930 nm is focused by an objective so as to produce a spot on

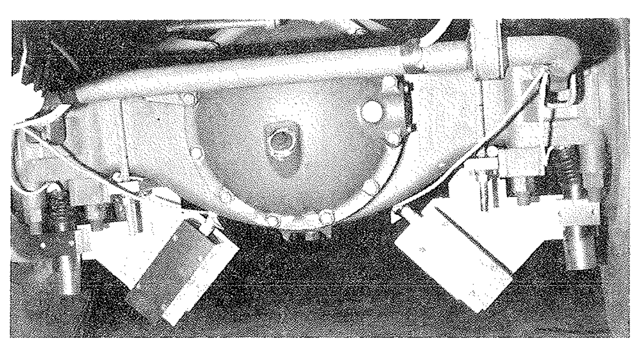
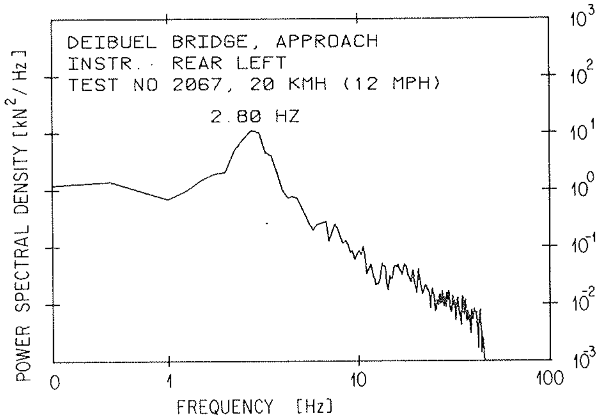


FIGURE 14 Two opto-electronic systems for measuring the dynamic wheel loads mounted on the rear axle of a test vehicle.

the pavement surface of approximately 15 mm (0.6 in.) diameter. The essential component of the camera, which observes the position of the light spot, is a two-dimensional photodetector.

Electric currents that give a measure for the position of the center of the light spot are induced in the photodetector. These currents are transformed to voltages that can be stored on magnetic tape by the PCM-data-acquisition system, which is installed either in the driver's cabin or near the bridge being tested. Signal transfer from the moving vehicle to the stationary PCM-system is performed by a 2.45 GHz telemetry link. Hence, measurement signals stemming from vehicle and bridge are acquired and recorded synchronously. Figure 15 shows an example of the power spectral density of a measured dynamic wheel load. The four-channel dynamic wheel load measurement system was extensively used within the framework of a research program but has not yet been introduced to the standard dynamic tests on highway bridges.



Note: Horizontal axis: Frequency 0-100 Hz, 400 lines, logarithmic scale. Vertical axis: Mean-squared dynamic wheel load density, full scale = 10^3 kN^2/Hz (divide by 20 to convert to sq. kip/Hz), logarithmic scale.

FIGURE 15 Power spectral density (PSD) of a dynamic wheel load signal.

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