Design Provisions for Dynamic Loading of Highway Bridges

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ABSTRACT

The Ontario Highway Bridge Design Code (OHBDC) contains provisions for dynamic load and vibration that differ substantially from those of other codes. In these provisions it is considered that the dynamic effects of vehicles crossing highway bridges can still be described in terms of an equivalent static effect that is a fraction of the design vehicle load, but the magnitude of this effect is described in terms of the natural frequency of the structure rather than the span length. Few codes are based on a limit-state design philosophy for both design and evaluation. Accordingly, new provisions were required for OHBDC that adequately represent the random effects of the dynamic component of load as typical design and evaluation vehicles traverse a span. A review of existing code provisions for impact, a discussion of vehicle-bridge interaction, and dynamic tests of bridges carried out in the Province of Ontario during the past 25 years are provided. The results of the tests are presented and discussed in the context of a design code for highway bridges. Some existing provisions were found unconservative for structures having a first flexural frequency between 2.0 and 5.0 Hz. Calibration of the load factors for dynamic load allowance for a reliability-based limit-state design code is described. In summary, the dynamic response of modern bridges to modern vehicles is reviewed and how this response can be catered to in a design code is described.

Investigations of the static and the dynamic responses of bridges to loading by both commercial and test vehicles have been part of routine test programs carried out by the Ontario Ministry of Transportation and Communications (MTC) in Canada during the past two to three decades. Investigations of dynamic behavior have been directed toward the response of new forms of construction for intermediate-span and long-span structures and assessment of pedestrian reaction to vehicles crossing flexible structures.

This test experience, together with a trend toward limit-state design for both bridge evaluation and bridge design, led to the development of the Ontario Highway Bridge Design Code (OHBDC). The OHBDC, first published in 1979 and revised in 1983, is a limit-state document. Development of the OHBDC by MTC required an almost complete evaluation of current design procedures. In particular, new provisions were required to represent the random nature of the dynamic component of load as representative design vehicles traversed a span or spans of a structure.

Old and current provisions for dynamic load allowance, the term favored here for impact, are reviewed. This review shows that many different dynamic load allowances have been used in design and that it is not clear that traditional non-limit-state codes model the physical behavior of vehicle-bridge interaction. The process by which the dynamic load allowance provisions of the OHBDC were developed is presented together with the test evidence and code provisions. The OHBDC provisions are believed representative of the principal characteristics of typical vehicle-bridge systems and include recognition of quasi-resonance between vehicle and bridge.

A HISTORY

A first step in the development of the OHBDC was to assess whether design provisions currently in use in North America and elsewhere for highway bridge dynamic loading were appropriate. This was based not only on a survey of those provisions but also on consideration of their derivation and intent.

Allowances for dynamic load, customarily referred to as impact factors, used by several countries are shown in Figure 1 in terms of span. There is general agreement that the allowance should be higher for short spans and should decrease as the longer span increases.

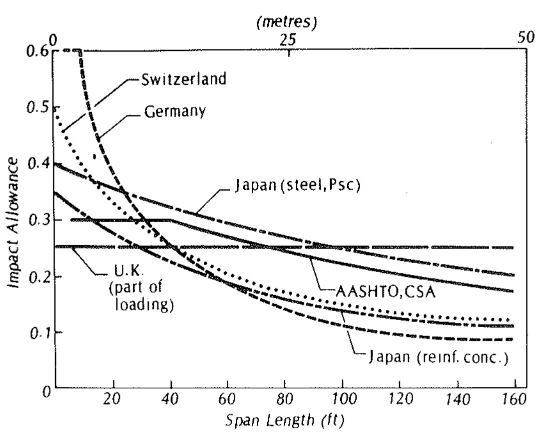


FIGURE 1 Typical impact provisions.

The development of impact factors in North America is of interest. In 1910, Thomson (1) suggested an impact stress allowance having the following form:

$$IS = (LLS)^2/2(DLS + LLS) \tag{1}$$

where

IS = impact stress due to live load,
DLS = stress due to static dead load, and
LLS = stress due to static live load.

The physical background leading to the design equation was neither given nor referenced, but it should be noted that the ratio of live-load to dead-load stress is the main parameter in the design equation rather than span.

North American highway bridge impact provisions were derived from railway engineering, where designers were required to recognize the hammer-blow effect of steam locomotive drive wheels. This hammer-blow effect produces a sinusoidal force having a frequency proportional to locomotive speed and gives rise to large impactive forces. In 1922, the American Railway Engineering Association (AREA) adopted the following relationship (2):

$$I = 50/(L + 150) \tag{2}$$

where I is the impact factor, not to exceed 0.30, and L is the span length in feet. In the same year AASHO suggested (2) the following (L in feet):

$$I = (L + 250)/(10L + 500) \tag{3}$$

A joint conference committee of AREA and AASHO in 1927 adopted this form (2) (L in feet):

$$I = 50/(L + 125) \tag{4}$$

Thus, the main imput to highway bridge impact allowance was experience with railway bridges and steam locomotives.

The first thorough investigation of highway bridge dynamic loading was conducted from 1922 to 1928 by an ASCE committee (2). This committee identified that decks and deck support components had different response characteristics from those of main longitudinal members. An impact allowance of 0.25 was recommended for decks, and for main longitudinal members the committee suggested the following (L in feet):

$$I = 50/(L + 160) \tag{5}$$

with I not greater than 0.25. Test data were obtained from 10 bridges by using a 15-ton truck driving at speeds up to 15 mph. The recommendation for main longitudinal members included the statement: "Data are too meager to establish a relationship between impact and span." One of the main concerns at this time was the difference in response between vehicles having solid and those having pneumatic tires.

Major studies in the 1950s and 1960s included those carried out by the University of Illinois (3) and as part of the AASHO Road Test (4). These were both analytical and experimental studies and identified roughness and undulation of the riding surface and the approach and bridge as major contributors to the dynamic response of a bridge.

A speed parameter (α) associated with a smoothly rolling axle crossing a span was considered important:

$$\alpha = V/2Lf \tag{6}$$

where

V = truck speed (ft/sec),
L = span (ft), and
f = first flexural frequency (Hz).

In addition, the ratio of axle spacing to span length was found significant. This work at Illinois achieved significant agreement between analytical and observed results and identified the broad scope of the problem, especially for simple-span bridges (3). Three-span continuous bridges were also examined and found to be more complex than simple spans. Neither a quasi-resonance effect of close truck and bridge frequencies nor torsional responses were noted.

An alternative impact factor was suggested as a consequence of this work (5,6):

$$I = 0.15 + \alpha \tag{7}$$

The first term represents the effect of initial oscillation of the truck entering the span and the second the effect of a smoothly rolling mass crossing the span. This form was not, however, adopted in any design code.

Computer simulations in the early 1970s resulted in a rather complicated set of impact factors for the various components of horizontally curved steel bridges (7). This appears to be the only addition to the familiar Equation 4, adopted in 1927 and still widely used some 57 years later (8,9).

The AASHTO specifications (8) use the impact factor to increase member stresses, not to increase loads, although it is not unusual for Equation 4 to be used in design offices as a factor to increase loads rather than member stresses.

This brief survey has shown that the provisions used for dynamic loading of highway bridges are based on early railroad and highway experience. Structures and materials in use then were not typical of current construction. Vehicles were also quite different from typical heavy highway loads currently legal in Ontario and other provinces of Canada, where up to 63,500 kg (630 kN) may be carried on an eight-axle vehicle having a length of 21 to 23 m (10).

Even in the 1920s, when these provisions were developed, a clear relationship between span and impact was not evident. Nevertheless, the AASHTO impact formula has not been unsatisfactory, at least from the point of view that few (if any) bridge failures can be attributed directly to dynamic response of the bridge.

Two consistent patterns emerge from the literature on dynamic loading of bridges. First, the problem is too difficult and complex to address in the context of a design code by analytical means. Second, test data are difficult to obtain and difficult to interpret in a manner relevant to the design provisions of a code. Perhaps, therefore, these considerations have contributed to an apparent lack of need for change in dynamic loading provisions.

VEHICLE-BRIDGE DYNAMIC INTERACTION

To appreciate the design of bridges for vehicles, a discussion of vehicle and bridge characteristics is of value. When a moving load crosses a span that is at rest, the span deflects from an equilibrium position. Forces acting on the span are a combination of those due to the vehicle and span masses. These forces combine to give maximum static and dynamic effects at or near the midspan in simple-span structures. The dynamic response of the bridge will be a combination of the flexural and torsional modes of vibration and a forced response associated with the load oscillating on its suspension system. Elastic resistance of the superstructure tends to restore the span to an equilibrium position, and frictional forces (damping) within the span dissipate energy transferred to the span by the moving load. A typical deflection-time trace for the midspan of a simple-span structure is given in Figure 2. This trace can be thought of as an influence line for deflection at the instrument location, the midspan point.

A steady force is applied to the riding surface by the tires of a vehicle traveling along a smooth

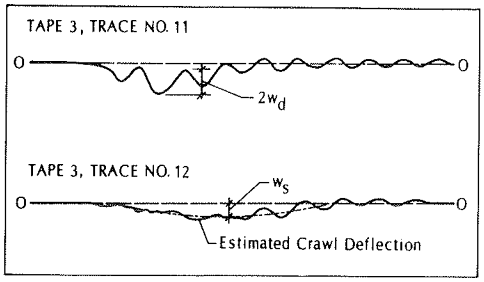


FIGURE 2 Typical deflection-time trace.

horizontal rigid riding surface at a constant velocity. This is an idealized situation, and the application of any external force caused by wind, steering, or braking will result in a change of applied tire force to the riding surface, as will variations in the profile of the riding surface. As a vehicle crosses a bridge superstructure, the superstructure deflects and further variation in vehicle axle load occurs. The instantaneous deflection of the superstructure is a function of the position of the vehicle, the previous deflection history, and the axle load variation. The vehicle and superstructure are inseparably coupled [Figure 3 (11)].

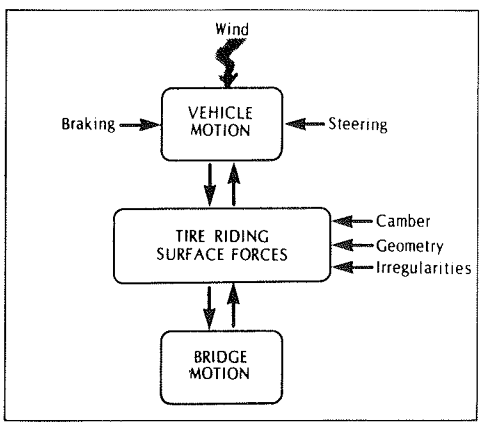


FIGURE 3 Vehicle and bridge interaction.

Thus, any description of the dynamic response of the vehicle-bridge system should include at least the mass distribution, natural frequencies, modes of vibration, and damping characteristics of the bridge superstructure; mass and dynamic characteristics of the vehicle; initial conditions of both vehicle and structure, including vertical displacements and velocity; and riding surface profile. Undulations and irregularities in the approach riding surface caused by repair, weathering misaligned expansion joints, snow, and ice all influence the initial condition of the vehicle. Camber variations, settlement, temperature-induced curvature, and badly maintained surfacing will also affect the superstructure riding surface profile. In addition, the superstructure may not be at rest because of other vehicles on or off the span. All the quantities noted previously cannot be easily monitored or measured within the normal limitations of budgets for either analyses or field tests.

Notwithstanding the complexity of the problem, simple models of vehicles and bridges can be used to gain insight into the principal vehicle and bridge characteristics governing response.

A single axle traversing a simple span without riding surface irregularities and response so small that the load is negligibly different from the static value corresponds to a point force crossing at constant velocity (V). The span deflection is increased over the static value by an amount dependent on the speed parameter α (Figure 4). For typical highway bridges and legal highway speeds, the speed parameter α is in the range 0.08 to 0.20, and the ratio of maximum to static deflection is bounded above by $[(1 + \alpha)/(1 - \alpha)]$ for all α. For typical bridges it is bounded above by $(1 + \alpha)$ (12-15).

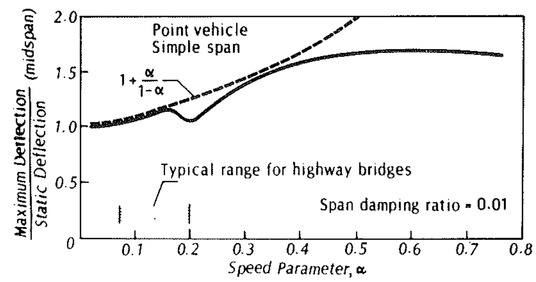
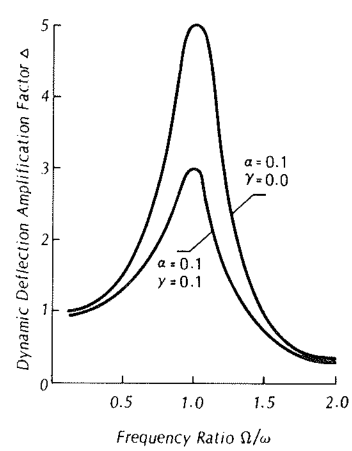


FIGURE 4 Simple-span dynamic amplification for moving-point load.

Now consider a constant force P combined with a constant-amplitude oscillatory load Q of circular frequency Ω, so that a force $(P + Q \sin\Omega t)$ traverses a simple-span bridge. This force represents an upper bound on the real situation because irregularities of the riding surface excite vehicle vibration but no energy is absorbed by the vehicle suspension. The dynamic deflection amplification factor (Δ) is strongly dependent on the ratio of vehicle frequency (Ω) to bridge fundamental frequency $(\omega = 2\pi f)$, as shown in Figure 5 for two levels of bridge damping in terms of the fraction of critical damping (γ). The response (Figure 5) is akin to resonance of a system with a single degree of freedom (12) but is not infinite for zero damping



Total Dynamic Amplification of Deflection $= 1 + Q\Delta/P$

FIGURE 5 Dependence of dynamic amplification on frequency ratio and damping.

because passage of the load limits the time the bridge is exposed to the force (13). This large amplification of deflection when the load frequency (Ω) corresponds to the bridge frequency (ω) can be thought of as quasi-resonance.

There are other parameters that affect bridge dynamic response to vehicle passage, such as the ratio of live load to dead load, tire stiffness, and suspension stiffness (11-16). These simple models examine vehicle and bridge dynamic interaction for the first mode of a simple span. Continuous and multigirder bridges may have several modes with frequencies close together. Vehicles have heave frequencies of 2 to 5 Hz, so it is likely that one of the lower modes of any bridge with a span of 20 to 25 m is in the same frequency range. For longer spans for which the first mode of the bridge is below 2 Hz, coincidence of vehicle frequency and frequency of a higher mode of the bridge is likely (17), as is the amplification of response.

In summary, dynamic amplification increases with speed for a moving force but decreases with speed for a sprung mass without damping. The initial conditions for even the simplest case of a moving force and mass entering a span are uncertain, and the initial conditions for a sprung vehicle entering a bridge are even more difficult to assess.

1956-1957 TEST SERIES

A group of 52 bridges known to vibrate was selected for test (18,19). A variety of differing types, spans, and cross-sectional geometry was chosen. Approach and deck conditions varied widely and included marked irregularities or undulations.

From the more than 2,000 individual records of bridge motion for the calibration and other vehicles, it was possible to obtain vehicle speed and axle spacing, maximum static deflection for a given vehicle, amplitude of vibration, and frequencies of vibration. The stiffness of the structure was calculated from the calibration-vehicle data and was used to obtain an equivalent load of all other vehicles. The equivalent load of a vehicle is related through an unknown load-distribution and axle-spacing function to the calibration vehicle.

The importance of the ratio of maximum dynamic deflection (w_d) to maximum static deflection (w_s) in dynamic response studies (Figure 2) is well known. Hence the ratio of maximum dynamic deflection (amplitude) to equivalent load, referred to as the amplitude factor, was used to obtain the amplitude developed by a vehicle of unit equivalent load.

Typical results of interest are shown in Figures 6 and 7 in terms of amplitude factor versus speed and vehicle load, respectively. Figure 6 indicates that the amplitude factor is a function of speed and can have a form associated with a constant moving mass traversing a structure. The influence of increasing equivalent weight on amplitude factor is clearly shown in Figure 7.

The frequency of vibration of the loaded structures was generally the first longitudinal flexural frequency, suitably corrected for the additional mass of the vehicle, or a forced frequency the value of which ranged from 2 to 3 Hz. This range corresponds to the bounce frequency normally reported for heavy commercial vehicles sprung only by tires and having an inactive suspension system (4). Among the various correlations attempted, the tendency of the median amplitude factor to decrease with static stiffness (not shown) and to increase for bridges with observed frequencies in the range of the 2 to 5 Hz was apparent (Figure 8) (18).

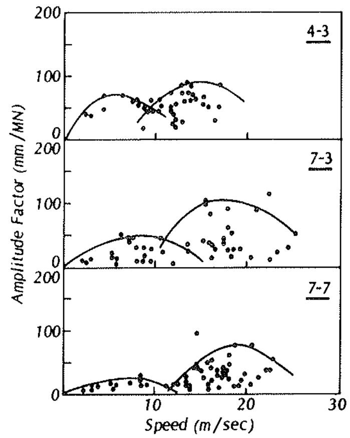


FIGURE 6 Amplitude factor versus speed.

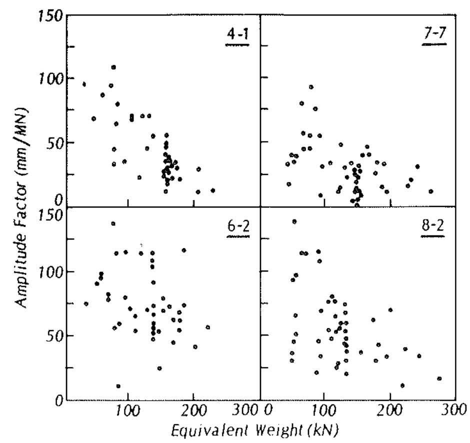


FIGURE 7 Amplitude factor versus equivalent weight.

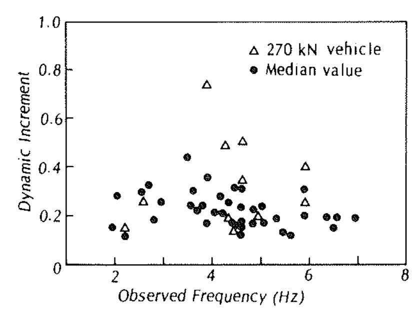


FIGURE 8 Dynamic increment versus observed frequency.

1969-1971 TEST SERIES

During this period a series of tests was completed on continuous concrete bridges by using a five-axle tractor-trailer combination weighing 400 kN (20). Calculated frequencies corresponded with observed values in most cases. The maximum observed dynamic amplification of deflection varied from 0.30 to 0.85 for bridges in the measured frequency range of 2 to 5 Hz because of a single test vehicle.

The observations from this test series led to the design concepts used for the Conestogo River Bridge

(21) in which by relaxing the live-load deflection to span requirements of AASHTO (8), it was possible to provide a distribution of longitudinal stiffness that yielded a first flexural frequency outside the quasi-resonance range associated with 2 to 5 Hz. The importance of the frequency content of the loading function on the magnitude of dynamic effects is clearly illustrated in Figure 9 (21) in which the footfall frequency of a horse drawing a buggy produced a greater dynamic response than a heavy truck did. The latter, however, produced the larger static response.

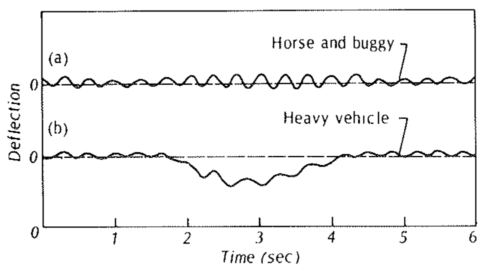


FIGURE 9 Deflection-time trace, sample span.

1980 TEST SERIES

The 1980 tests were carried out to ensure that values of mean dynamic response and the associated coefficient of variation used in calibration of the OHBDC were representative of modern vehicles and bridges. Test results indicated that reductions could be made in the values specified for dynamic effects in the first edition of OHBDC (22) as part of the second edition (23).

A total of 27 structures were selected at 22 locations, 5 of which were twin structures. They included 14 steel spans of 22 to 122 m, 10 concrete spans of 16 to 41 m, and three timber spans of about 5 m (24). The approaches, expansion joints, and deck of all of these bridges were in good to excellent condition.

Four test vehicles were used. TV1 and TV2 were five-axle tractor-trailer combinations having gross weights of 391 and 414 kN, TV3 was an eight-axle combination having a gross weight of approximately 580 kN, and TV4 was a three-axle service vehicle (241 kN). The vehicles are representative of heavy commercial vehicles operating in Ontario and all were loaded close to their legal limit.

More than 100 individual runs were recorded for each bridge by both test vehicles and normal traffic crossing the spans at a variety of speeds. In addition, the response of the bridge to truck passage was assessed subjectively by technicians associated with the tests; they used the Reiher-Meister scale (25).

The FM tapes of acceleration values recorded during the test were used to determine frequencies, mode shapes, and damping ratios of the bridge vibration modes (26). Between 6 and 12 modes of vibration of longitudinal flexure, torsion, and transverse flexure could normally be identified with certainty for longer-span continuous bridges (Figure 10). In contrast, the three timber bridges tested did not appear to have any vibration modes.

Values of the first flexural frequency are shown against the longest span of the bridge in Figure 11. Although there is a clear trend, with only a few data points covering a diversity of construction, it is unreasonable to suggest a simple relationship between frequency and span that could be

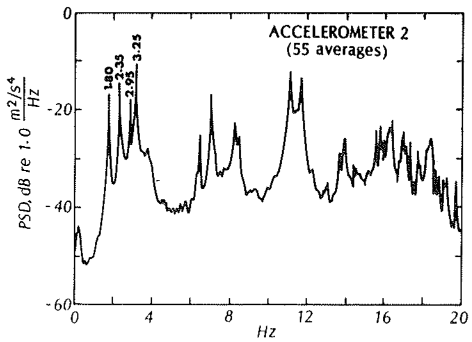


FIGURE 10 Power spectra, Gull Lake.

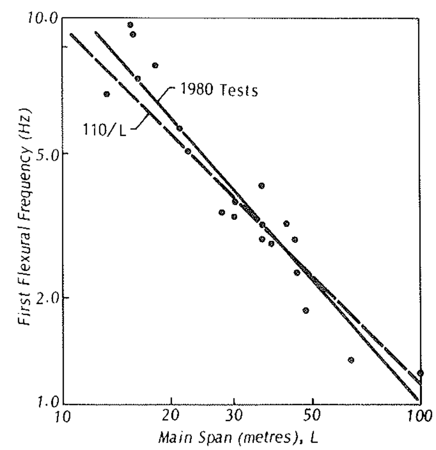


FIGURE 11 Frequency versus main span.

codified. A simple relationship such as $f = 110/L$ (meters) appears to be useful for the preliminary design estimate of frequency. The test series did indicate that all components of the structure resist the action of both static and dynamic load.

By using the typical responses shown in Figure 12 for a three-span continuous bridge, three response

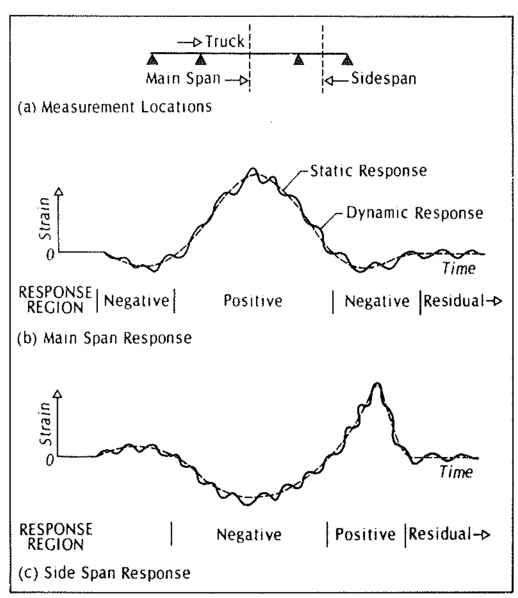


FIGURE 12 Typical responses.

regions were defined. Dynamic amplifications were computed for the three response regions by using the greatest static response for any instrument location. The overall statistics of dynamic amplification are presented in Table 1. These data include for each bridge all single-truck runs by test vehicles and by other traffic at all speeds in any one lane. The mean dynamic amplifications are not large, even though some individual test dynamic amplifications greater than 0.5 were observed. The coefficients of variation are large, varying between 0.56 and 1.11 with a mean of 0.85. The data of Table 1 show that the mean dynamic amplifications of continuous bridges are approximately equal for both positive and negative regions of the influence line for deflection at a given point.

Test vehicles TV1 and TV2 were similar in overall dimensions and weight. TV2 had an air suspension, whereas TV1 had leaf springs. The mean dynamic amplification by TV2 was about 60 percent of that for TV1 for all runs on all bridges. Presumably the air-spring and parallel shock-absorber suspension system provides damping under all conditions, whereas the leaf-spring assembly only absorbs energy for large displacements or high rates of loading (4).

The mean dynamic amplification for all runs generally decreased with increase in weight of trucks for spans greater than about 30 m (Figure 13). This reduction is presumably because additional axles are required for an increase in legal gross vehicle weight, and these axles are not in phase, which moderates the dynamic effect. If the product of truck weight and mean dynamic amplification is used as a measure of total dynamic load associated with a vehicle, Figure 14 shows the dynamic load for various test vehicle weights, corresponding to the data of Figure 13. The dynamic load for each of the four bridges shown is sensibly constant for each test vehicle weight; the different magnitudes are associated with different pavement irregularities for each bridge.

HUMAN RESPONSE

During a test technicians and others were asked to stand on the bridge and provide a subjective rating of bridge vibration caused by passing trucks. The Reiher-Meister descriptors (25) were used: not perceptible, slightly perceptible, distinctly perceptible, strongly perceptible, disturbing, and very disturbing. No training or calibration was given. The threshold of perception was found to be in the range of 0.015 to 0.025 g. The slightly, distinctly, and strongly perceptible ratings had mean accelerations of 0.039, 0.052, and 0.076 g, respectively. For one structure with a measured frequency of 0.75 Hz, the highest and mean of observed accelerations under normal traffic were 0.062 and 0.036 g, respectively. This particular structure had a live-load deflection to span ratio nearly twice that permitted by AASHTO (8).

OHBDC deflection criteria for pedestrian service-

TABLE 1 Overall Statistics of Dynamic Amplification

Bridge No.[a]	Positive Region Mean	Positive Region CV	Negative Region Mean	Negative Region CV	f (Hz)	Location[b]
1	0.129	0.67			4.00	
4	0.069	0.74			3.13	
6	0.136	0.90			5.00	
7	0.057	1.00			10.63	
8	0.110	1.11			12.00	
9	0.305	0.91				
10	0.093	0.84				
11	0.156	0.98			10.38	
12	0.077	0.65			3.13	
13	0.098	1.04			8.06	
14	0.150	0.85	0.105	1.18	7.13	a
	0.119	0.69	0.033	0.88		b
15	0.068	0.61	0.006	1.00	5.88	a
	0.031	0.72	0.003	0.76		b
16	0.161	0.72	0.134	0.79	3.31	c
	0.205	0.77	0.104	0.98		d

Bridge No.[a]	Positive Region Mean	Positive Region CV	Negative Region Mean	Negative Region CV	f (Hz)	Location[b]
17	0.164	0.70	0.123	0.42	2.94	a
	0.141	0.72	0.100	0.57		
18	0.191	0.55	0.192	0.56		
19	0.174	0.56	0.171	0.48	1.80	c
	0.112	0.60	0.084	0.66		d
20	0.194	0.76			2.31	
21	0.210	0.93			2.88	c
	0.167	0.82				e
23	0.177	1.03	0.137	0.85	3.63	
24	0.236	1.05	0.204	0.78	2.69	
26	0.079	0.73	0.097	0.63	3.44	a
	0.062	0.83	0.041	0.85		b
27	0.090	0.63	0.092	0.52	0.75	a
	0.099	0.67	0.061	0.66		b
	0.084	0.59	0.075	0.55		d

Note: CV = coefficient of variation, f = mode frequency.

[a]See report by Billing (24).

[b]Location: a = main span, b = side span, c = midspan, d = support, e = floor beam.

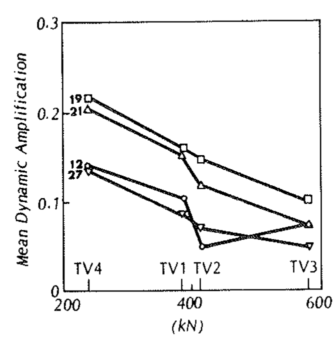


FIGURE 13 Mean dynamic amplification versus test vehicle weight.

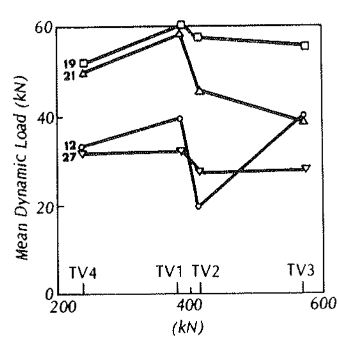


FIGURE 14 Mean dynamic load versus test vehicle weight.

ability are compared with results obtained from tests in Figure 15; observed deflections are scaled to provide values appropriate to a serviceability limit-state truck load. None of the bridges tested had significant pedestrian use. However, the criteria appear appropriate even though the bridges in the field behaved in such a way as to include the stiffening effects of curbs and barrier walls.

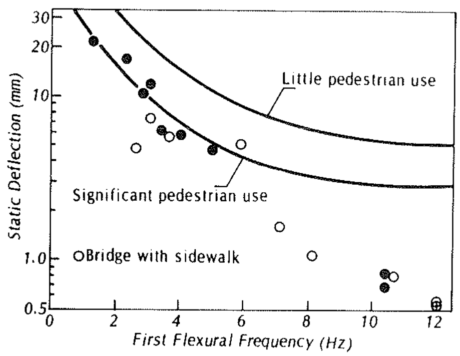


FIGURE 15 Pedestrian serviceability.

DESIGN OF OHBDC PROVISIONS

The OHBDC was undertaken to implement into design and evaluation various recent research findings regarding structural design and response (27). It was also undertaken to make design loads representative of actual and legal heavy truck traffic in the Province of Ontario (10).

The existing provisions for dynamic loading of bridges were elementary and familiar to design engineers, so they were easy to apply. However, as also noted previously, the dynamic components of loads arise from a complex process of vehicle and bridge dynamic interaction. It was therefore likely that any provisions that would require a significant increase in computation or complexity for what was often only a few percent of the total design load would not be accepted readily by design engineers. The question of whether the impact formula could or should be retained was carefully considered. It became apparent that change was not only necessary but essential, for three reasons. First, continued use of a formula bearing little relation to observed behavior of bridges in some span ranges would inhibit future editions of OHBDC and other code developments. Second, the need for a realistic representation of bridge loading becomes more important as analysis methods improve and as bridges become more slender and fatigue more important to design. Finally, because OHBDC was to be one of the first codes in North America developed by using an approach based on limit-state reliability, if any substantive change was to be made to the dynamic loading effects, it should be made with the first edition of that code. Once the need for change was realized, it was possible to focus on the task of developing a form and values for the provisions that would jointly satisfy the designer's need for simplicity and adequately represent the main dynamic effect of vehicle loads.

The literal interpretation of the term "impact factor" was considered too narrow to express the complex interaction of vehicle components, undulation and roughness of approach and bridge riding surface, bridge dynamics, and vehicle speed. It was discarded in favor of the term "dynamic load allowance."

The OHBDC provisions on dynamic loading were written with future developments in mind. The designer was therefore permitted to use any approved dynamic analysis or test or both to develop a dynamic load allowance. In lieu of these, which would probably only be in special circumstances, a dynamic load allowance was prescribed as an increase to and a fraction of the prescribed highway live load. This contrasts with other codes in which dynamic effects are accounted for by an increase in stresses in designated components and members caused by the live load (8,9).

This change means that the components of a bridge need not be defined with respect to their load-carrying function, because the appropriate allowance will automatically be included in the load applied to a particular component. It also represents the process actually used by designers.

The principal dynamic loading provisions of the second-edition OHBDC (23) are as follows:

1. The dynamic load allowance for a single wheel or axle unit of the OHBDC truck, shown in Figure 16, shall be 0.4. This allowance will be used primarily for design of deck slabs, short-span floor beams, and other components governed by the local effects of the impactive action of wheel load.

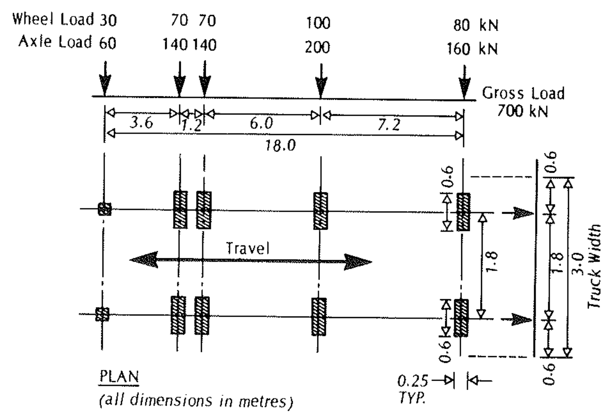


FIGURE 16 OHBDC truck.

2. The dynamic load allowance for more than a single axle of the OHBDC truck acting on a structure having no span in excess of 22 m shall be 0.3. This allowance will be used primarily for design of simple and continuous spans, transverse floor beams, and diaphragms where strong interaction between truck and bridge is unlikely. Typically, a span of 22 m would have a frequency no less than 5 Hz.

3. For a structure having any span greater than 22 m, the dynamic load allowance for the truck portion of the lane load shall depend on the first flexural frequency of the bridge, as shown in Figure 17. The dynamic load allowance for the uniformly distributed portion of the lane load shall be 0.1. This allowance will be used for design of main longitudinal members of the bridge where significant response of the bridge modes of longitudinal and torsional vibration is likely.

4. The dynamic load allowance for soil-steel structures shall be 0.4 for zero cover, reduced as depth of cover increases.

5. The dynamic load allowance for timber bridges shall be reduced by a factor of 0.7. This recognizes the higher damping of this type of construction.

6. For evaluation a reduced dynamic load al-

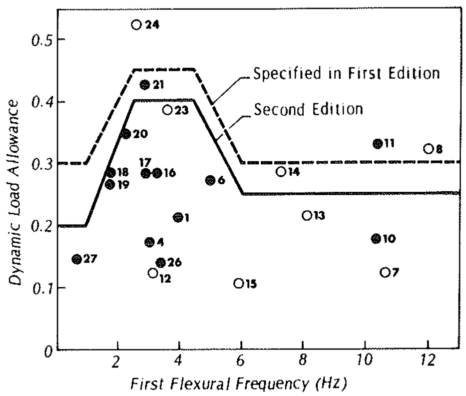


FIGURE 17 Dynamic load allowance versus frequency.

lowance shall be used for passage by a single vehicle carrying an exceptional load at low speed.

The values just given apply to loading in a single or multiple traveled or design lane or lanes. Multiple presence factors for dynamic loading in more than one lane were taken as 0.7 for two lanes, 0.6 for three lanes, and 0.5 for four or more lanes, and these factors were incorporated in the multiple presence factors for highway live load; this disguised the reduction in the dynamic load allowance due to multiple presence but facilitated calculation.

The values for dynamic load allowance given earlier are those that resulted from code calibration (28). Calibration was a process carried out to define load factors that would result in a reasonably uniform safety index (β) for all members of all bridge types. The load factor accounts for uncertainty in load and analysis and may include a professional factor. Although there are significant differences in longitudinal and transverse distributions of live load and dynamic load and in their variability, it was decided that a common load factor should be used for both live and dynamic loads as a convenience.

$$\alpha_{LL} I = \alpha_I \bar{I} \tag{8}$$

where

- α_{LL} = specified live load factor, also to be used for dynamic load;
- I = specified dynamic load allowance;
- α_I = computed load factor for dynamic load based on $\bar{I}$, and
- $\bar{I}$ = mean dynamic amplification

Therefore

$$I = \alpha_I \bar{I}/\alpha_{LL} \tag{9}$$

A typical value of α_I might be 2.5, and α_{LL} was specified as 1.4; hence a specified dynamic load allowance of 0.4 (say) would result from a mean observed dynamic amplification of only 0.22.

Values of mean dynamic amplification obtained from the tests (Table 1) were scaled by using Equation 9 and appropriate ratios of load factors (α_I/α_{LL}) ranging from 1.6 to 2.2 , depending on the coefficient of variation (26) and the results plotted in Figure 17. The high dynamic amplification present for the majority of bridges in the region of 2 to 5 Hz is captured through the OHBDC provisions. AASHTO values at 2 and 3 Hz would be 0.16 and 0.20, respectively, for typical spans.

SERVICEABILITY

Some codes retain limitations on the ratio of depth to span of main longitudinal members and of deflection to span (8,9) introduced by railroad engineers during the 19th century. A review of these limitations in 1958 was unable to establish a basis for the limitations nor was change recommended (29).

The OHBDC considered specific provisions covering span-depth and span-deflection limitations but noted that such limitations might inhibit future innovation in design. Finally, because deflection was not regarded as a limit state, these limitations were discarded in favor of bridge vibration as a serviceability limit state. Pedestrians on a bridge are sensitive to acceleration of the superstructure produced by passing vehicles. Because it is difficult for the designer to compute accelerations, they were transformed to equivalent deflections at the edge of the structure, assuming average truck weights and bridge dynamic response. Three levels of vibration control were identified, and the two lower deflection levels are presented in Figure 15. The upper level (not shown) applies to bridges without sidewalks, which would be traversed only by maintenance personnel. The lowest level might apply to bridges in cities or in rural regions where they might be used for viewing or fishing. The second level is for bridges with sidewalks where few pedestrians are expected. Data from tests show that even the lowest level will generally be unrestrictive for spans greater than 20 m (26).

DISCUSSION

The recent report by the ASCE Committee on Loads and Forces on Bridges (30) focused attention on the problem of impact on bridges. Some assistance may be offered by this paper based on Ontario's experience in the resolution of the research problems identified by that committeee. The dynamic response of a vehicle subsequent to traversing an undulating approach and irregular expansion joint is unlikely to be quantified for use in the analysis of vehicle-bridge interaction. The mean value of the undulating component of vehicle load is influenced by vehicle suspension systems and vehicle length and appears to be sensibly constant for a given bridge and approach condition (Figure 14); it is not a function of gross vehicle weight for spans longer than the vehicle. The importance of suspension system characteristics and riding surface to the mean dynamic load associated with vehicles is not new (4-6,15,18). The structural engineer has little or no control over either suspension or riding surface properties and hence must rely on the results of field observations.

The Ontario studies, even as early as the mid-1950s, indicated that dynamic amplification values for individual bridges with frequencies in the region of 2 to 5 Hz were on the average greater than the similar mean values for bridges with spans of 15 to 20. This trend (Figure 11) is also apparent in the 1968-1971 data (not shown) (20) and in the 1980 data (Figure 17), and it is not unreasonable to consider that a dynamic property of a structure should be a primary variable in the dynamic response. There is no reason, even noting the variability of dynamic amplification, to expect this trend to be only an Ontario phenomenon, and indeed, as recent Swiss data (31) (Figure 18) illustrate, it is not. The Swiss data, for maximum response on smooth pavements, are from new bridges proof-tested by using similar vehicles (160 kN) traveling along the longitudinal centerline of the structure. The amplification values are bounded by a line having the same

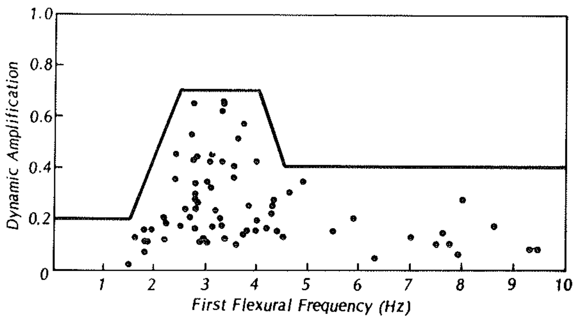


FIGURE 18 Peak dynamic amplification versus frequency, Swiss results.

form but not the same numerical values as those for the OHBDC (Figure 17).

Thus it does appear that first flexural frequency should be considered as a major variable for dynamic load allowance, particularly for spans with frequencies less than 5 Hz. Although the frequency range (2 to 5 Hz) may appear troublesome for design, no attempt should be made to avoid this frequency range, because dynamic load allowance values are available. On the other hand, designs can be created to reduce the large allowances associated with quasi-resonance (21).

The question of the need to consider dynamic effects at the ultimate limit state and what value should be used is for the calibration experts. Perhaps future design codes will incorporate an allowance for dynamic effects directly into the design loads at the ultimate and serviceability limit states and provide appropriate models for the analysis of vehicle effects at both limit states that reflect both static and dynamic response characteristics.

CONCLUSIONS

The provisions regarding dynamic loading of highway bridges in use in North America have been essentially unchanged for more than 50 years. It is questionable whether these provisions were representative of bridge behavior at the time they were developed. They are certainly not representative for large, heavy trucks on bridges that are becoming more slender and of longer span in the interests of economy.

The OHBDC has developed a new terminology and form for dynamic loading. The code attempts to represent the principal mechanisms of bridge loading and response in a manner that the designer can use with little change from current methods. The provisions have been written so as not to restrict further developments and are essentially independent of design vehicle geometry. All provisions of the 1983 OHBDC are discussed in another paper in this Record (Dorton and Bakht).

The OHBDC provisions have been built from the experience of the past by retaining values of dynamic load allowance for short spans but have added to this experience by considering the impactive manner of single-axle loads acting on the riding surface and vehicle-bridge interaction over a wide range of bridge types. The parameters used in the provisions refer to a dynamic characteristic of structure, frequency, rather than just span. The change in form of dynamic loading as a function of frequency has been supported by tests completed by others.

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Notice: The conclusions presented in this paper are those of the authors and not necessarily those of any sponsor.

Implementation of the Analytical Capabilities Required for the Aseismic Design of Bridges

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ABSTRACT

The design of a highway bridge located in a region of high seismic risk must include a detailed and accurate analysis of the bridge to determine its maximum anticipated seismic loads. To comply with newly developed code requirements and to ensure the utmost confidence in the predicted response, the seismic analysis should be performed by using the appropriate analytical procedures. The recently developed computer program Seismic Analysis of Bridges (SEISAB) used to conduct seismic analyses that comply with both the current AASHTO specifications and the Applied Technology Council seismic design guidelines is described. In addition, a description is given of the single-mode spectral method developed for the new guidelines for a specific category of bridges with low to moderate seismic vulnerability. An example is included to demonstrate the applicability of this method to a two-span bridge. A second example is included to illustrate how SEISAB-I was used to conduct a response spectrum dynamic analysis on a six-span curved bridge. Included also is a description of the nonlinear dynamic analysis capabilities to be included in the next version, SEISAB-II. The implementation of SEISAB-I through workshops funded by the National Science Foundation and the acceptance of the program based on trainee evaluations are also briefly described.

Both the current AASHTO bridge specifications (1), which were upgraded following the 1971 San Fernando earthquake, and the more recently adopted AASHTO Seismic Design Guidelines for Highway Bridges (2) require that a single-mode or multimode response spectrum analysis be conducted in the design of bridges to be located in zones of higher seismic activity. Because the analytical procedures involved in seismic analyses are new to many bridge de-