Dynamic Testing of Highway Bridges—A Review

B. BAKHT AND S. G. PINJARKAR

A review is presented of the recent technical literature in the English language dealing with bridge dynamics in general and dynamic testing of highway bridges in particular. It is shown that several definitions have been used for the impact factor and thus the same set of field test data may lead to widely varying estimates of the impact factor. In nearly all the reviewed references, there is little or no justification for using a particular definition, suggesting that each of the various definitions was regarded as axiomatic, requiring no justification. It is also shown that there are additional factors that may be responsible for misleading conclusions from the test data; these factors include vehicle type, vehicle weight, transverse position of the vehicle with respect to the reference point, differences in dynamic increment of strains and deflections, presence of bearing restraint forces, and roughness of the riding surface. A preferred method of interpreting the field test data for obtaining a representative value of the impact factor is suggested. It is shown that the impact factor is not a tangible entity susceptible to deterministic validation.

Vehicles that are expected to cross a highway bridge during its lifetime are accounted for in the design of the bridge through a statistically applied design load and a certain prescribed fraction of it, which is traditionally referred to as the impact factor, and lately as the dynamic load allowance. The static design load is a tangible entity that can be derived from the static weights of actual and foreseen vehicles in such a way that the load effects induced by it in any bridge are representative, with a certain degree of reliability, of the load effects induced by these actual and foreseen vehicles. The impact factor, on the other hand, is an abstract entity that is supposed to account for the magnification of load effects in a bridge caused by the interaction of the vehicle and the bridge.

Despite its abstract nature, the impact factor has been used in the design of bridges for several decades. There have been numerous attempts to measure this elusive quantity in bridges through dynamic field testing. The purpose of this paper is to review, and draw some general conclusions from, the technical literature dealing with bridge dynamics in general and dynamic testing of highway bridges in particular. It is noted that this paper is a summary of a more detailed report (*1*).

DEFINITION OF IMPACT FACTOR

As long ago as 1931, it was suggested that the *impact increment of dynamic force* be defined as the amount of force, expressed as a fraction of the static force, by which the dynamic force exceeds the static force (*2*). Recognizing that the impact increment of dynamic force is not necessarily the same as the *impact increment of stress*, the latter was defined as the amount of stress, expressed as a fraction of static stress, by which the actual stress as a result of moving loads exceeds the static stress.

Researchers interpreting test data from dynamic load tests have often used the term *dynamic increment* for the same quantity that has been defined by Fuller et al. (*2*) as the impact increment of stress or that could have been defined as the impact increment of deflection. However, there is no uniformity in the manner by which this increment is calculated from test data. The different ways of calculating the dynamic increment can be explained conveniently with the help of Figure 1, which has been constructed from data of an actual dynamic test with a two-axle vehicle on a right simply supported plate girder bridge (*3*). This figure shows the variation of both the dynamic and static deflections at midspan of a girder with respect to time. The dynamic deflections were obtained when the test vehicle traveled on the bridge at normal speed, and the static deflections were obtained when the vehicle traveled at crawling speed so as not to induce dynamic magnification of deflections. Figure 1 also shows the median deflections that were obtained by averaging consecutive peaks of dynamic deflections. As can be seen in this figure, the median deflections are not the same as the static deflections; however, a numerical procedure for filtering out the dynamic portion of the response can give the median responses that are fairly close to the static responses.

It may be noted that a fictitious scale of deflections has been introduced in Figure 1 in order to facilitate the explanation regarding the interpolation of the test data. Regarding this figure, a notation is now introduced, as follows:

δ_{stat} = maximum deflection under the vehicle traveling at crawling speed.

δ_{dyn} = maximum deflection under the vehicle traveling at normal speed. This deflection is also denoted as δ_{max}.

δ'_{stat} = maximum deflection obtained from the curve of median deflections. Note that δ_{stat} and δ'_{stat} do not necessarily take place at the same load location.

δ_{min} = minimum dynamic deflection in the vibration cycle containing δ_{max}.

δ_1 = static deflection corresponding to δ_{max}. As may be seen in Figure 1, δ_1 is not necessarily the maximum static deflection.

δ_2 = median deflection corresponding to δ_{max}.

B. Bakht, Structures Research Office, Research and Development Branch, Ministry of Transportation, 1201 Wilson Avenue, Central Building, Downsview, Ontario, Canada M3M 1J8. S. G. Pinjarkar, Raths, Raths and Johnson, Inc., 835 Midway Drive, Willowbrook, Illinois 60521.

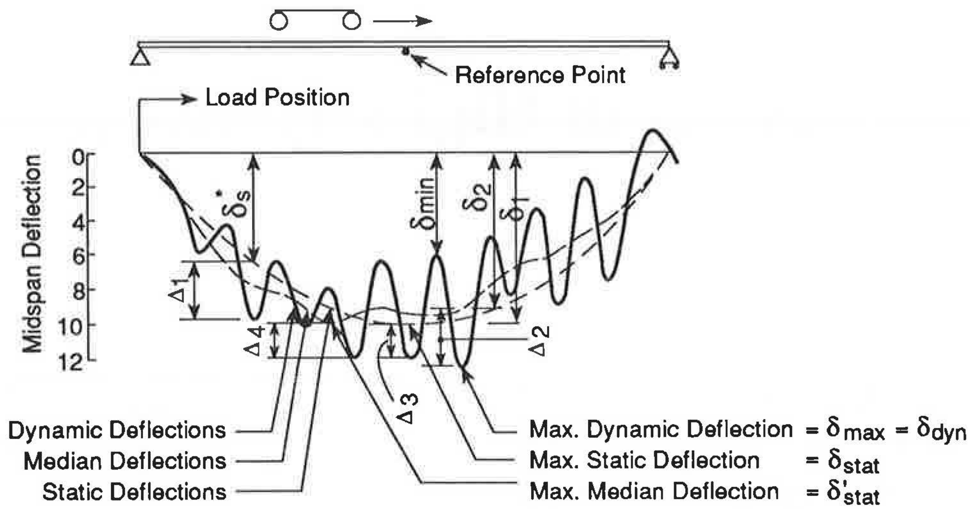

FIGURE 1 Midspan deflections under a moving vehicle load.

δ_s^* = static deflection at the same location where Δ_1 is recorded.

Δ_1 = maximum difference between dynamic and static deflections; as shown in Figure 1, Δ_1 does not necessarily take place at the same load position that causes either δ_{stat} or δ_{dyn}.

Δ_2 = maximum difference between dynamic and median deflections.

Δ_3 = difference between dynamic and static deflections at the same load location that causes δ_{stat}.

Δ_4 = difference between dynamic and median deflection at the same load location that causes δ'_{stat}.

These definitions can be more general if the word *deflection* is replaced by *response*.

From the data plotted in Figure 1, the various deflection parameters are found to have the following values according to the fictitious scale from which units are deliberately omitted to maintain generality: δ_{stat} = 10.0; δ_{dyn} = δ_{max} = 12.3; δ'_{stat} = 9.9; δ_{min} = 6.2; δ_1 = 9.1; δ_2 = 9.8; δ_s^* = 6.4; Δ_1 = 3.2; Δ_2 = 3.1; Δ_3 = 2.0; and Δ_4 = 2.0.

The various definitions are now described, many of which have been used in the past to obtain from the test data the dynamic increments, or similar parameters given different names. For the sake of convenience, all these different parameters will henceforth be referred to generically as dynamic amplification factors and will be denoted by the symbol *I*.

Definition 1

According to the definition of impact increment of dynamic response by Fuller et al. (*2*), the largest of *I* would be given by

$$I = \Delta_1/\delta_s^* \tag{1}$$

Using this expression, the deflection data of Figure 1 would lead to I = 0.500. It should be noted, however, that this method is the result of a hypothetical and impractical extrapolation of a definition, which, perhaps, was not intended for this purpose. This method has not been used to interpret test data in any of the references that were studied for this state-of-the-art report.

Definition 2

A commonly used variation of Definition 1 is that *I* is taken as the ratio of the measured instantaneous dynamic response to the maximum static response. Thus,

$$I = \Delta_3/\delta_{stat} \tag{2}$$

This definition of *I* has been used in the interpretation of data from several dynamic tests (e.g., *3–5*) and in nearly all analytical studies (e.g., *6–9*). According to Definition 2, the value of the impact factor obtained from the data given in Figure 1 is 0.200.

Definition 3

When the static deflections are assumed to be the same as median deflections, Definition 2 of *I* changes to

$$I = \Delta_4/\delta'_{stat} \tag{3}$$

This definition gives *I* the value of 0.202. It is noted that this definition does not seem to have been used by any of the references cited in this paper.

Definition 4

Definition 4 was used in Switzerland to interpret test data from the dynamic bridge tests conducted during 1949 to 1965 (*10*). According to this definition, the dynamic increment *I* is given by

$$I = \frac{\delta_{max} - \delta_{min}}{\delta_{max} + \delta_{min}} \tag{4}$$

It is noted that this definition of the dynamic increment was abandoned in Switzerland after 1965 in favor of Definition 5 which follows. However, this definition has been used in New Zealand until fairly recently (*11,12*).

Definition 5

According to the fifth definition, which has been used in Switzerland for tests conducted before 1945 and after 1965 (*10*), the dynamic increment I is given by

$$I = \frac{\delta_{dyn} - \delta_2}{\delta_2} \tag{5}$$

Using this definition, the data in Figure 1 give $I = 0.255$.

Definition 6

A variation of Definition 5 would be when the static response corresponding to the maximum dynamic response is taken as the same as the median response obtained from the dynamic test data. In this case, I is given by

$$I = \frac{\delta_{dyn} - \delta_1}{\delta_1} \tag{6}$$

leading to a value for I of 0.352. This definition has been extensively used to interpret the results of many dynamic tests on bridges in Ontario (e.g., *13–16*).

Definition 7

For interpreting the data from some dynamic tests conducted in Ontario (*17,18*), the following expression was used for obtaining I:

$$I = \frac{\delta_{dyn} - \delta'_{stat}}{\delta'_{stat}} \tag{7}$$

For the specific case under consideration, the value of I is then found to be 0.242.

Definition 8

If the actual static responses are used instead of median responses, the following variation of Equation 7 is obtained:

$$I = \frac{\delta_{dyn} - \delta_{stat}}{\delta_{stat}} \tag{8}$$

This definition gives I a value of 0.230 for the data in Figure 1.

COMPARISON OF VARIOUS DEFINITIONS

Table 1 contains the values of I obtained by the various definitions from the same set of data. It can be seen in this table that the values of I are all different and range from 0.2 to 0.5.

In nearly all of the references studied for this paper, there is little or no discussion or justification for using a particular definition of the dynamic amplification factor. This seems to suggest that each of the various definitions was regarded as being axiomatic and requiring no justification. Yet the variety of results given in Table 1 confirm that the definition of I is far from axiomatic. What can be regarded as axiomatic, however, is the definition of the amplification factor for the response at a given instant. According to this definition, $I = \Delta/\delta_s$, where Δ is the difference between the static and dynamic responses at the instant under consideration, and δ_s is the corresponding static response.

The axiomatic definition of amplification factor is used, justifiably, in all of the analytical studies; it is, however, of little use in bridge design because its value changes with time and load position. What is required for design purposes is a single value of the amplification factor with which maximum dynamic response can be computed from the maximum static response, so that

$$\delta_{dyn} = \delta_{stat}(1 + I) \tag{9}$$

The values of δ_{dyn} obtained by using δ_{stat} of 10.00 and the values of I given by the various definitions are also given in Table 1. Ideally, the use of the amplification factor according to Equation 9 should have returned the same value of δ_{dyn}

TABLE 1 VALUES OF DYNAMIC AMPLIFICATION FACTOR I AND COMPUTED MAXIMUM DYNAMIC DEFLECTION

Definition No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Value of I	0.500	0.200	0.202	0.330	0.255	0.352	0.242	0.230
Value of δ_{dyn} from Eq. (9)	15.00	12.00	12.02	13.33	12.55	13.52	12.42	12.30

Note: Maximum value of measured dynamic deflection is 12.30.

that was measured in the field, that is, 12.30. It can be seen in Table 1 that none of the definitions has given the correct value of δ_{dyn}, except Definition 8, the equation for which (Equation 8) is, in fact, the same as Equation 9, which gives the correct value of δ_{dyn}. It is interesting to note that the apparently logical Definition 8 has not been used in any of the references studied for this state-of-the-art study, a complete list of which is given in a previous paper by Bakht and Pinjarkar (*1*).

At a cursory glance, it may seem strange that such a variety of definitions emerges from the process of selecting a single governing value from the values of I that are obtained by an axiomatic definition. A careful scrutiny of the problem will readily reveal that the diversity in the definitions of I from measured responses is the consequence of the fact that (a) the static response of a bridge is not necessarily the same as the median response obtained from the dynamic test data and (b) the maximum static and dynamic responses do not always take place under the same load position (e.g., *3*). If the static and median responses were identical and the maximum static and dynamic responses took place simultaneously, the diversity of definitions of I would disappear and Definitions 2 through 8 would all give the same value of I for a given set of data.

FACTORS RESPONSIBLE FOR MISLEADING CONCLUSIONS

The technical literature reports a fairly large scatter in the values of the dynamic amplification factor of a given response, even when the bridge and the vehicle are the same. From these observations it can readily be concluded that the dynamic amplification factor is not a deterministic quantity. To obtain a single value of this factor for design purposes, it is necessary, as is shown later in the paper, to know the statistical properties of the scatter of data, in particular the mean and variance of the amplification factor. The various parameters that can influence the statistical properties of the amplification factors computed from the test data are discussed in the following subsections. If not accounted for carefully, these parameters can influence misleadingly the way in which the measured data are interpreted.

Vehicle Type

It is already known that the dynamic amplification factor for a bridge is influenced significantly by the dynamic characteristics of the vehicle with respect to those of the bridge. Despite this fact, most dynamic tests on bridges have been conducted with specific test vehicles. The data from such tests cannot, for obvious reasons, be regarded as representative of actual conditions. The amplification factors obtained from tests with only specific test vehicles can provide only a qualitative insight into the problem of bridge dynamics. They should not be used to obtain the final single value of the impact factor that is to be used in calculations for design or evaluation. A representative value of the impact factor can be calculated realistically only when data are gathered under normal traffic and over relatively long periods of time.

Vehicle Weight

Several researchers have concluded from observed data that the dynamic amplification factor resulting from a vehicle decreases with the increase of vehicle weight (e.g., *2*, *11*, *12*, *17*). It can be appreciated, in light of this information, that the amplification factors corresponding to lightly loaded vehicles, which are irrelevant to the design load effects, are likely to weigh the data unduly on the higher side. The data corresponding to lightly loaded vehicles should not be used at all in the calculation of the impact factor, unless, of course, the impact factor is sought specifically for lighter vehicles, as it may be for the evaluation of the load carrying capacity of existing substandard bridges.

Vehicle Position with Respect to Reference Point

The cross section of a three-lane slab-on-girder bridge is shown in Figure 2. The bridge has five girders, all of which are instrumented for dynamic response measurement; it carries a vehicle in the far right-hand lane so that Girders 4 and 5 carry the vehicle load directly. In this case, Girders 1 and 2, being remote from the applied load, will carry a very small portion of the static load. Yet the dynamic amplification of the small portion of the static load carried by these two girders is likely to be fairly large. It has been observed by several researchers that the dynamic amplification factor at a reference point well away from the load can be larger than that for a reference point directly under the load. Clearly, the former amplification factor has no relevance as far as the maximum static load effects are concerned at the cross section of a bridge.

A parameter, α, is used by Cantieni (*10*) to define the relative position of the vehicle with respect to that of the reference point; this parameter is defined by

$$\alpha = \frac{D}{H + 1.25} \tag{10}$$

where D is the distance (in meters) in the transverse direction between the reference point and the nearest line of wheels, H is the depth of construction of the bridge (also in meters) at the instrumented cross section, and the number 1.25 in the denominator is the half-width of the vehicle in meters. When α is less than 1.0, the reference point is assumed to lie within the direct influence zone, in which case the dynamic test data are considered to be relevant. In a case when α is greater than 1.0, the data are ignored, being of no consequence.

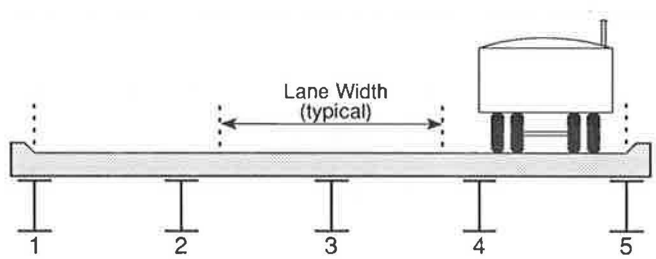

FIGURE 2 Cross section of a three-lane bridge.

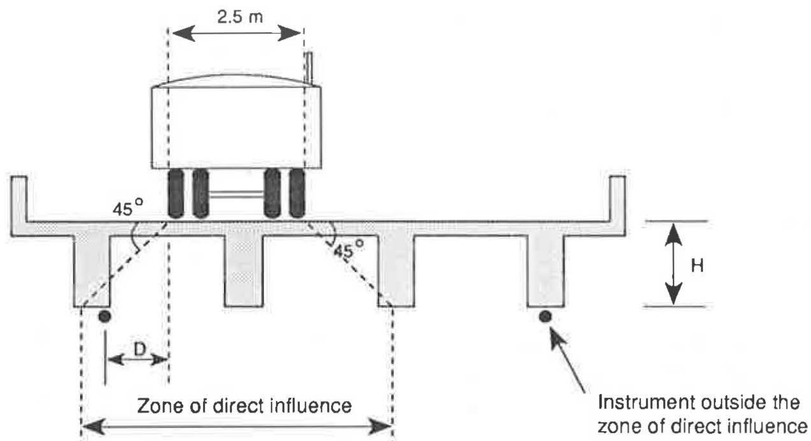

FIGURE 3 Definition of the zone of direct influence (*10*).

Figure 3 identifies the zone of direct influence defined by Cantieni (*10*). It can be seen that this zone is obtained simply by drawing 45 degree lines on the bridge cross section from the outer line of wheels of the test vehicle. This limit on the zone of influence cannot be regarded as being applicable to all bridges because it does not take into account the transverse load distribution characteristics of the bridge. It may be more appropriate to take into consideration only the data from that reference point at which the static load response is the maximum across the cross section.

The statistical properties of the dynamic amplification factor, computed from the test data, can be regarded as realistic only if the extraneous data from outside the zone of influence are excluded from consideration. It is surprising that the attempt to exclude such extraneous data has been explicitly mentioned in only a few of the references reviewed for the study at hand.

Deflection Versus Strain Measurements

The response of a bridge in dynamic tests is conventionally measured by means of vertical deflections. The measurement of strains is resorted to only when the measurement of deflections is found to be difficult (e.g., *19, 20*). The dynamic amplification factors obtained by both approaches are assumed to be applicable with equal validity to all responses including deflections, bending moments, and shear forces. Strictly speaking, the practice of regarding the amplification factors as being applicable for other responses is not correct. It has been demonstrated convincingly by the AASHO test (*4*) that, under similar conditions, the dynamic amplification factors computed from measured deflections are always greater than the corresponding factors computed from measured strains. The same conclusion has also been reached by others (e.g., *18*).

Bearing Restraint Effects

With the exception of AASHO tests (*4*), the measured frequencies of tested bridges were nearly always found to be much greater than the calculated frequencies. This observation leads to the conclusion that the flexural stiffness of bridges is nearly always greater than the calculated stiffness. The reason for this apparent discrepancy has been attributed, wrongly, in many of the references to (a) a very high modulus of elasticity of concrete; (b) interaction with the main components of secondary components such as horizontal bracings and barrier walls; and (c) the presence of a high degree of composite action between the girders and deck slab, even when they do not have any mechanical shear connection between them.

Unintentional composite action in an apparently noncomposite bridge can also not always be regarded as a major factor to stiffen the bridge. This is so because even those slab-on-girder bridges, which have mechanical shear connectors between the deck slabs and girders and in which full composite action has been considered in the calculations, have been found stiffer than shown by calculations.

The effective flexural rigidity of the tested bridges by using both the measured midspan static load deflections and the measured frequencies were calculated by Biggs and Suer (*3*). When it was found that the two procedures gave different flexural rigidities, it was concluded that, "although this cannot be completely explained, it is possible that the causes are a greater participation of secondary elements and a greater degree of frictional restraint at the ends of the span." This is, perhaps, the first time in recent literature that attention has been paid to the presence of bearing restraint effects as being the possible cause of bridges being stiffer than shown by calculations. Recent tests have confirmed that fairly large bearing restraint forces develop in slab-on-girder bridges at the interfaces of girders and the surfaces they rest on (*21–23*). These large bearing restraint forces have been found to stiffen the bridges appreciably.

The observed frequencies of composite steel and prestressed concrete bridges tested in the AASHO study (*4*) were in good agreement with the calculated values that were obtained by using the measured cross sections and other properties of the actual bridges. It is noted that, in these bridges, the girders were supported by rocker-type bearings, which prevented the development of substantial bearing restraint forces.

The presence of bearing restraint forces induces compressive strains in the bottom flanges of the girders; consequently, the girder strains do not remain directly proportional to the girder moments. In such cases it is doubtful whether dynamic amplification factors obtained from girder strains can apply exactly to girder moments. The inclusion of bearing restraint forces in the interpretation of measured dynamic strains is still not a practical proposition. This problem, however, still needs a careful study.

MISCELLANEOUS OBSERVATIONS

Quality of the Riding Surface

It is mentioned in nearly every reference dealing with bridge dynamics that the roughness of the riding surface of a bridge and its approaches has a significant influence on the dynamic magnification of load effects in the bridge. It is usual in dynamic bridge tests to account for the riding surface irregularities by placing a wooden plank of appropriate thickness in the path of the test vehicle. As might be expected, the dynamic amplification factors corresponding to such tests are always larger than those that correspond to similar tests without the wooden plank.

The practice of performing dynamic tests by creating a temporary irregularity in the riding surface of even well-maintained bridges has been defended on the basis that even a well-maintained bridge can have a sudden irregularity introduced in the riding surface, for example, as a result of an object being dropped accidentally from a vehicle or the accumulation of packed snow. It should be noted, however, that in bridges with more than one lane, the design loading for failure almost invariably corresponds to the rare event of exceptionally heavy vehicles being simultaneously present in two or more lanes of the bridge. The probability of such a rare event happening, at the same time as the formation of an accidental irregularity in the riding surface of an otherwise well-maintained bridge, is indeed so small as to be negligible.

The commentary to the second edition of the Ontario Code (*24*) suggests that a high value of the dynamic load allowance should be used if the approach is likely to remain unpaved for extended periods of time, or if the expansion joints between the superstructure and approach pavements are not expected to be flush with the roadway. Taking a cue from this suggestion, it may be appropriate to test such bridges by placing a plank in the path of the test vehicle. For well-maintained bridges, on the other hand, there does not seem to be any justification for adopting this approach.

Multilane Loading

It has been observed in many references that the dynamic amplification factor for more than one vehicle is always less than that for a single vehicle, and the out-of-phase dynamic actions of the various vehicles are usually cited as the reason for this phenomenon. It seems appropriate that the value of the impact factor should decrease with the increase in the number of loaded lanes. The commentary to the Ontario Code in its first edition (*25*) specified a separate set of multipresence reduction factors for the dynamic load allowance (DLA). These factors were much smaller than the corresponding factors for static loading. The separate reduction factors for DLA were abandoned in favor of the same factors for both the static loading and DLA in the second edition of the Code. It has been shown by Jaeger and Bakht (*26*) that the multipresence factor for combined static and dynamic loading, m_f, is approximately given by

$$m_f = \frac{m_{fs} + m_{fd}\,DLA}{1 + DLA} \tag{11}$$

where m_{fs} and m_{fd} are the multipresence reduction factors for static and dynamic loadings, respectively.

RECOMMENDED PROCEDURE

From a survey of the technical literature dealing with dynamic testing of highway bridges, a preferred procedure emerges naturally for obtaining, through a test, a representative value of the impact factor for single vehicles that can be used realistically in the load capacity evaluation of an existing bridge. The various steps involved in this preferred and recommended procedure are given in the following subsections.

Instrumentation

For obtaining the dynamic amplification factors for longitudinal moments, it is preferable to measure strains rather than deflections. In slab-on-girder bridges, strains can be measured conveniently at the bottom flanges of the girders, even if the girders are of concrete, in which case special strain gauges or strain transducers may have to be used. Care should be exercised in selecting a crack-free zone for instrumentation of concrete components.

Calibration Test

It is desirable to perform both static and dynamic tests on the bridge with vehicles of known weights and configurations. The static test can be performed either under a stationary vehicle positioned at preselected locations or under a vehicle moving at crawling speed of less than about 10 km/hr. If the latter procedure is adopted, it may still be necessary to filter out the dynamic responses in order to obtain the static load responses. However, in this case the filtered responses can be expected to be very close indeed to the actual static load responses.

Elimination of Extraneous Data

At a given instrumented cross section, the data corresponding to only a single reference point should be considered when computing the dynamic magnification factor resulting from a vehicle pass on the bridge. This reference point should be the one at which the maximum static, or median, value of the response is recorded for the vehicle pass under consideration.

As discussed earlier, the dynamic amplification factors corresponding to light vehicles, being relatively on the high side, tend to bias the data. It is, therefore, necessary that the data considered for developing the statistics of the dynamic amplification factor correspond to the weight class of the design or evaluation vehicle. This can be achieved as follows.

The maximum response at a reference point resulting from the design or evaluation vehicle can be readily obtained by extrapolation of the data obtained from the calibration tests. Let this maximum response be denoted as ε. As shown in Figure 4, the observed maximum static load response can be divided into a number of strata, with each stratum representing the vehicles of a certain class of weight that relates to the load effects in the bridge rather than the gross vehicle weight. It is recommended that the observed data from the dynamic test should be divided into various groups, depending upon the division of the maximum static load effects, of the kind shown in Figure 4. For example, the dynamic amplification factors corresponding to the design or evaluation vehicle should be obtained only from that data for which δ'_{stat} lies within 0.9ε and 1.1ε.

Method for Obtaining Dynamic Amplification Factor

From a purely logical standpoint, the appropriate definition for computing the dynamic amplification factor from measured responses would appear to be Definition 5, described earlier. However, this definition requires that a bridge be tested under the same vehicle separately for both dynamic and static effects. Such a requirement is obviously not realistic

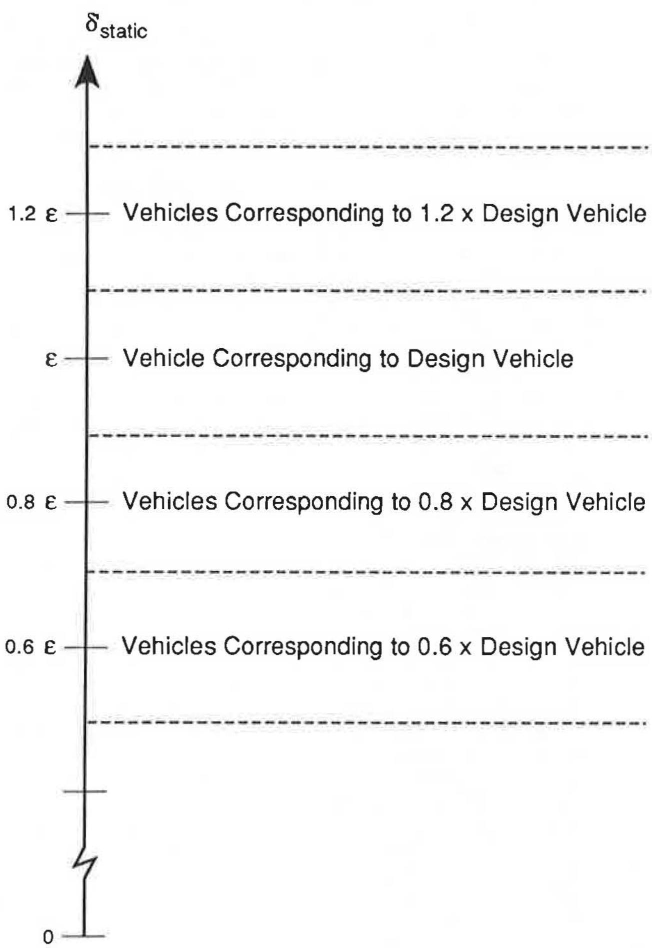

FIGURE 4 Scheme for classifying vehicles.

when data are being collected under normal traffic. In such a case, it is suggested that the next best definition is Definition 7. This definition can give fairly reliable results, especially if it is found from the calibration test that the maximum static load response δ_{stat} is close in magnitude to the corresponding maximum median response δ'_{stat}. The case in which the two maximum responses are significantly different from each other is rather rare. For such bridges, Definition 7 can still be used but only after making appropriate adjustments for the difference between δ_{stat} and δ'_{stat}.

It is noted that when a vehicle is longer than the length of the influence line of the instrumented component, the static response at a reference point resulting from the moving vehicle may not be smooth, that is, it may have "static oscillations." For such cases, the obtaining of the median responses by automatic filtering is made especially difficult when the period of static oscillations matches with the period of dynamic oscillations.

Calculation of Impact Factor

The term *impact factor* is used here to denote that single value of the dynamic amplification that is used in the calculations for the design or evaluation of the bridge. It is noted that, as mentioned earlier, the impact factor is also referred to as the *dynamic load allowance*.

Simply because of the scatter in their values, the dynamic amplification factors computed from the dynamic test data cannot be used directly as the impact factor. Neither should an upper-bound value of the amplification factor be used as the impact factor, because this is likely to prove overly conservative. A logical approach to computing a representative value of the impact factor from the test data would be to cater for the variability of the amplification factor in the same way as is done for the variability of the static loads. One procedure proposed in the commentary to the second edition of the Ontario Code (*24*) and reported by Billing (*17*) can be used to achieve this goal. According to this procedure, the specified value of the impact factor, I_s, depends not only on the statistics of the amplification factor but also on the live load factor specified in the Code and the safety margin to which the Code is calibrated. The expression for obtaining I_s is as follows:

$$I_s = \frac{\bar{I}(1 + vs\beta)}{\alpha_L} \tag{13}$$

where

$\bar{I}$ = mean value of the dynamic amplification factor,
v = coefficient of variation of the dynamic amplification factor, that is, the ratio of standard deviation and mean,
s = the separation factor for dynamic loading, which has been found to have a value of 0.57,
β = the safety index, which typically has a value of about 3.5 for highway bridges, and
α_L = the live load factor.

It is noted that when the distribution of mean largest vehicle weights is log normal, it may be more appropriate to use the following expression for obtaining the specified value of this impact factor:

$$I_s = \frac{\bar{I}}{\alpha_L} e^{v s \beta} \tag{14}$$

The use of the live load factor in Equations 13 and 14 requires some discussion. The use of the live load factor in determining the specified value of the impact factor is open to question, as the live load factor alone does not define the variability of the static loads. This variability is accounted for by both the live load factor and the specified static loads. For example, it is possible to get the same live load effects by simultaneously doubling the live load factor and halving the specified static loads. In this case, the specified value of the impact factor would be reduced incorrectly if Equations 13 and 14 were used.

Ideally, α_L, in Equations 13 and 14, should account for the variability of static loads, rather than being equal to the live load factor. It is recommended that, in the absence of more rigorous analysis, the value of α_L should be taken as 1.4, which is also the live load factor as specified in the Ontario Code (*24*). Using the numerical values of the various variables, Equations 13 and 14 can be written as

$$I_s = 0.71\,\bar{I}\,(1 + 2.0v) \tag{15}$$

$$I_s = 0.71\,\bar{I}\,e^{2.0v} \tag{16}$$

It should be noted that for $\bar{I}$ and v to be representative of actual conditions, there should be a sufficient number of values of the amplification factor obtained from the test data.

CONCLUSIONS

From a survey of the technical literature dealing with the dynamic testing of highway bridges, it has been found that there is a general lack of consistency in the manner in which the test data are interpreted to obtain the values of the dynamic amplification factor. A preferred method of calculating these factors has been proposed. It has been shown that the impact factor is not a tangible entity susceptible to deterministic evaluation; it can be accounted for in the design and evaluation of bridges only by a probabilistic approach. Despite being obtained from field data by a preferred procedure, the impact factor still remains an abstract entity that should, in general, be treated only as a design convenience. A recommended procedure for obtaining a design value of the impact factor by testing a highway bridge under dynamic loads has been developed.

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Publication of this paper sponsored by Committee on Dynamics and Field Testing of Bridges.