State-of-the-Art Review of Dynamic Impact Factors of Highway Bridges

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Abstract: Dynamic impact of moving vehicles on bridges is an important and long-standing issue in the design and evaluation of bridges and has received much attention from researchers and engineers. The use of the dynamic impact factor (IM) to account for the impact effect of vehicles has been widely accepted in bridge engineering. Accurate evaluation of the IM will lead to safe and economical designs of new bridges and provide valuable information for condition assessment and management of existing bridges. Nevertheless, agreement on the evaluation of IMs is yet to be reached. Numerous studies have shown that the evaluation of the IM is a difficult task because it is influenced by a large number of parameters and uncertainties. As a result, different forms and values of IMs are specified by different bridge design codes and this disagreement has been debated in many studies in the past few decades. Furthermore, some field tests observed that the IMs in design codes are overestimated while many other field tests have suggested that code provisions may lead to underestimation of IMs, indicating the need to develop a more accurate assessment method for IMs. It is the objective of this paper to review and summarize the important methodologies and findings of the study of the dynamic IM of highway bridges conducted over the past two decades. While reviewing the advances achieved in the past two decades, much effort was made to identify the remaining controversies and gaps left in this field. Therefore, it is hoped that this review can also provide necessary background information for researchers and engineers to further examine these problems and help identify future research directions in this field. **DOI:** 10.1061/(ASCE)BE.1943-5592.0000672. *© 2014 American Society of Civil Engineers.*

Author keywords: Impact factor (IM); Dynamic load allowance (DLA); Dynamic amplification factor (DAF); Vehicle-bridge interaction (VBI); Bridge code; Field test; Analytical study; Parametric study.

Introduction

Vehicle-induced vibration of highway bridges is one of the primary concerns in bridge engineering that has been long recognized. It is now well-known that moving vehicles will exert a dynamic impact effect on bridges, namely, the increment from the static load effect. To account for such an effect, a dynamic impact factor (IM) is typically proposed in design practice and the total live-load (LL) effect is usually calculated as

$$\mathrm{LL} = (1 + \mathrm{IM}) \times R_{\mathrm{sta}} \tag{1}$$

where R_{sta} = static-load effect; and $(1 + \mathrm{IM})$ represents the dynamic amplification for the static load effect.

The dynamic IM plays a vital role in the practice of bridge design and condition assessment. Accurate evaluation of IMs will lead to safe and economical designs for new bridges and provide valuable information for condition assessment and management of existing bridges. However, the evaluation of an IM is a rather complicated issue because of the sophisticated mechanism of the vehicle-bridge interaction (VBI) and a large number of parameters influencing IMs, including the dynamic characteristics of both the bridge and the vehicle, road surface condition, vehicle speed, traffic flow condition, etc. Some of these parameters are too specific to address or to predict. Such complexity is inconsistent with the requirement of simplicity by the bridge design codes. As a result, various bridge design codes give different expressions for IMs. For example, the AASHTO (1992) standard specifications define an IM as a function of bridge span length; the Ontario code [Ontario Ministry of Transportation (OMT) 1991] specifies the IM based on the number of axles of the vehicle. Nevertheless, many studies have shown that these code provisions have failed to provide accurate estimations of IMs to some extent (e.g., Huang et al. 1993; Liu et al. 2002; Deng and Cai 2010).

Accordingly, numerous studies have been conducted to investigate the dynamic behavior of highway bridges under the impact of moving vehicles. However, no consensus on this subject has yet been reached. Many previous studies have yielded different or even contradictory findings. Paultre et al. (1992) presented an extensive review of early studies conducted on bridge dynamics and the evaluation of the dynamic amplification factor (DAF). McLean and Marsh (1998) provided a synthesis that summarizes the important knowledge and findings with respect to vehicular dynamic load effects on highway bridges. Recently, the development of theory and technology has enabled the application of more authentic models in analytical studies, which have significantly improved the accuracy for the calculation of bridge dynamic responses. Correspondingly, many more investigations have been carried out successively, adding important knowledge into this field.

Furthermore, recent years have seen a growing trend in the application of statistical and probabilistic approaches in the study of IMs. Some common approaches include probability plots (Hwang

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Note. This manuscript was submitted on March 2, 2014; approved on June 30, 2014; published online on July 25, 2014. Discussion period open until December 25, 2014; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Bridge Engineering***, © ASCE, ISSN 1084-0702/04014080(14)/$25.00.

and Nowak 1991; Kim and Nowak 1997; Kim et al. 2007), regression analysis (Chang and Lee 1994; Deng and Cai 2010), and reliability analysis (Deng et al. 2011). Also, to evaluate the IM on a statistical basis, various distribution types for the IM have been adopted, including the Gumbel distribution (Deng and Cai 2010), lognormal distribution (Kim et al. 2007), generalized extreme-value distribution (Caprani 2013), and normal distribution (Chang and Lee 1994).

This paper aims to present a comprehensive review of the recent analytical and experimental studies of the dynamic IM of highway bridges. The definition of the IM is first introduced and various design code provisions for the IM are presented. Then, the results from large-scale field tests are reviewed together with the instrumentation, test procedures, and data processing methods. A review of the models and methodologies adopted in analytical studies is also presented. Parametric studies focusing on the influences of different parameters on the IM are discussed in detail. Finally, conclusions are drawn based on the recent findings and suggestions are made for the future investigation of the IM.

Definition of the IM

Although several definitions of the dynamic IM have been given in the literature (Bakht and Pinjarkar 1989), the IM is generally defined as follows based on the maximum dynamic and static responses:

$$\mathrm{IM} = \frac{R_{\mathrm{dyn}} - R_{\mathrm{sta}}}{R_{\mathrm{sta}}} \tag{2}$$

where R_{dyn} and R_{sta} = maximum dynamic and static responses at a certain location on a bridge, respectively. This definition simply takes the maximum of both responses regardless of whether the two maximum responses occur simultaneously. OBrien et al. (2010) pointed out that such a definition of the IM is unnecessarily conservative because it fails to address the reduced probability of two maximum responses happening concurrently. Nonetheless, this definition is used in most studies because of its convenience of use and accordance with the design purpose (McLean and Marsh 1998). In addition, an equivalent definition known as the dynamic load allowance (DLA) is also adopted in practice. Another similar definition, the DAF, defined as the ratio of the maximum dynamic response to the maximum static response, is also used to represent the dynamic impact effect. The relationship between the three different terms can be expressed as $\mathrm{IM} = \mathrm{DLA} = \mathrm{DAF} - 1$.

To calculate the IM, the maximum dynamic response is usually obtained by taking the maximum value of the measured or predicted dynamic response; the derivation of the static response can be obtained from the following methods (Paultre et al. 1992): (1) carrying out a quasi-static test where vehicles move across the bridge at a crawl speed; (2) filtering the measured dynamic response with a low-pass filter to eliminate the dynamic components of signal; and (3) using FEMs to calculate the static response when the vehicle weight and loading position are known. The application of these methods will be discussed in the subsequent sections.

Furthermore, the calculated IMs based on different load effects vary, i.e., different load effects are amplified to different degrees because of moving vehicles. For example, many studies have reported that IMs calculated from displacements are larger than those from strains (e.g., Chang and Lee 1994; Humar and Kashif 1995; Huang 2001; Li et al. 2008; Szurgott et al. 2011), while other studies have reported contradictory findings (e.g., Fafard et al. 1998; Senthilvasan et al. 2002; Aluri et al. 2005). Nevertheless, some researchers believe that it is inappropriate to adopt IMs calculated from displacements to amplify internal forces in design practice (Wang et al. 1994; Huang et al. 1995b; Fafard et al. 1998). From this perspective, it seems irrational to assign a uniform value of the IM for all types of load effects. However, this phenomenon is neither emphasized nor differentiated in many design codes.

In addition, as an alternative to the DAF, the concept of the assessment dynamic ratio (ADR) was introduced by Caprani (2005). The ADR is defined as the ratio of the characteristic total load effect to the characteristic static load effect. The characteristic total and static effects are the expected maximums for the specified return period. For example, *Eurocode 1: Action on Structures—Part 2: Traffic Loads on Bridges* [European Committee for Standardization (CEN) 2003] is based on a return period of 1,000 years for new bridge designs, and characteristic total and static effects usually correspond to various loading scenarios instead of the same scenario used to obtain the DAF. Compared with the conventional DAF, which is unnecessarily conservative, the use of the ADR is a statistical approach based on extrapolation to calculate the lifetime dynamic allowance. OBrien et al. (2009) extrapolated the characteristic load effects from simulation results based on the extreme value theory (Castillo 1988; Getachew and OBrien 2007) and investigated the variation of the ADR with the return period. OBrien et al. (2010) used the previously developed model to simulate the traffic of 10,000 years such that the characteristic load effects can be interpolated. They found that the ADR is much smaller than the DLA specified in the design codes, implying the conservatism in the assessment of existing bridges. Caprani et al. (2012) presented a procedure using bivariate extreme-value statistical analysis to determine the ADR and also found very small values of the ADR.

IM in Bridge Codes

Many national bridge codes have specified different provisions for the IM. In this section, various national bridge code provisions for the IM are reviewed. The following code provisions of the DLA or IM are applied to bridge superstructures; for buried structures the specifications for the DLA or IM are usually in different forms.

AASHTO Code

The AASHTO (1992) *Standard Specifications for Highway Bridges* have specified the IM as a function of the bridge span length

$$\mathrm{IM} = \frac{15.24}{L + 38.10} \leq 0.3 \tag{3}$$

where L = bridge span length (in meters). The IM is applied to both truck and lane loads. In the AASHTO (1994) *LRFD Bridge Design Specifications*, the term DLA was used to replace the IM. The DLA is specified based on different limit states and components, which remains unchanged in the current AASHTO (2012) LRFD code, as shown in Table 1. Unlike the specifications in the standard specifications, the DLA is independent of the bridge span length and is applied to truck and tandem loads excluding lane load. In addition to bridge design, the AASHTO (1989) *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* specifies the IM

Table 1. DLA in AASHTO (2012)

Component	Limit state	DLA (%)
Deck joint	All limit states	75
All other components	Fatigue and fracture limit states	15
	All other limit states	33

as a function of road surface condition for load rating of existing bridges as shown in Table 2. From Table 2, it can be seen that larger IMs are assigned as the road surface condition worsens. In the AASHTO (2003a) *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, a DLA of 0.33 is specified for the strength and service limit states to account for the dynamic effects induced by moving vehicles. The value of 0.33 is deliberately conservative considering certain distressed approach and bridge deck conditions. Thus, for longitudinal members having spans greater than 12.19 m (40 ft) and less severe approach and deck surface conditions, the DLA may be reduced as suggested by Table 3. In the current AASHTO codes, the AASHTO (2011) *Manual for Bridge Evaluation* replaced the AASHTO LRFR code (AASHTO 2003a). However, the provision for the DLA still remains the same.

Ontario Code and Canadian Code

The *Ontario Highway Bridge Design Code* (OHBDC) [Ontario Ministry of Transportation and Communications (OMTC) 1983] specifies the DLA based on the first flexural mode frequency of the bridge, as shown in Fig. 1, where it can be seen that a higher DLA value is assigned for bridges with frequencies within 2–5 Hz. This is because this frequency range covers the common frequency of vehicle bouncing and this matching of vehicle and bridge frequencies may lead to quasi-resonance, inducing a large dynamic response on the bridge. The provisions for the DLA in the OHBDC (OMT 1991) are considerably different from the previous edition. The OHBDC (OMT 1991) specifies the DLA as a function of the number of axles of the vehicle, as shown in Table 4. This provision takes into consideration the fact that heavier vehicles, which usually have more axles, usually have lower DLAs.

In the *Canadian Highway Bridge Design Code* (CHBDC) [Canadian Standards Association (CSA) 2006], the DLA is also dependent on the number of axles, and is applied to the CL-W truck, which is an idealized five-axle truck used for the purpose of design. The provisions for DLA are basically the same as those specified in the OHBDC (OMT 1991). Also, a DLA value of 0.50 is specified for deck joints.

Table 2. IM in AASHTO (1989)

Wearing surface condition	Description	IM
Good	No repair required	0.1
Fair	Minor deficiency, item still functioning as designed	0.1
Poor	Major deficiency, item in need of repair to continue functioning as designed	0.2
Critical	Item no longer functioning as designed	0.3

Table 3. DLA in AASHTO LRFR Code (AASHTO 2003a) and *Manual for Bridge Evaluation* (AASHTO 2011)

Riding surface condition	DLA
Smooth riding surface at approaches, bridge deck, and expansion joints	0.1
Minor surface deviations or depressions	0.2

Chinese Code

The *General Code for Design of Highway Bridges and Culverts* [Ministry of Transport of the People's Republic of China (MTPRC) 1989] by the Chinese Ministry of Transport specifies the IM as a function of the bridge span length. For the main structural members of concrete bridges, the IM is calculated using the following expression:

$$\mathrm{IM} = \begin{bmatrix} 0.3 & L \leq 5\ \mathrm{m} \\ 0.3 \times (1.125 - 0.025L) & 5\ \mathrm{m} < L < 45\ \mathrm{m} \\ 0 & L \geq 45\ \mathrm{m} \end{bmatrix} \tag{4}$$

For the main structural members of steel bridges, the IM is specified as

$$\mathrm{IM} = \frac{15}{37.5 + L} \tag{5}$$

where L = bridge span length (in meters). However, in the 2004 edition of the *General Code for Design of Highway Bridges and Culverts* (MTPRC 2004) the IM is specified as a function of the fundamental frequency of bridges as follows:

$$\mathrm{IM} = \begin{pmatrix} 0.05 & f < 1.5\ \mathrm{Hz} \\ 0.1767 \ln f - 0.0157 & 1.5\ \mathrm{Hz} \leq f \leq 14\ \mathrm{Hz} \\ 0.45 & f > 14\ \mathrm{Hz} \end{pmatrix} \tag{6}$$

where f = fundamental frequency of bridges.

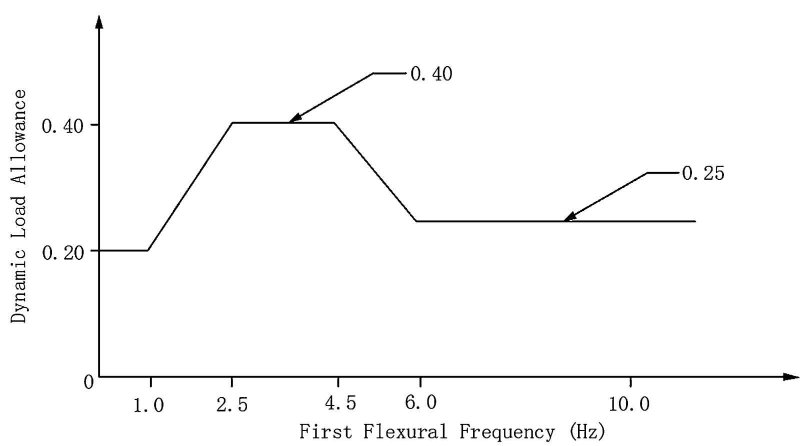

Fig. 1. DLA in OHBDC (data from OMTC 1983)

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New Zealand Code

In the New Zealand Transport Agency (NZTA 2013) *Bridge Manual*, the dynamic load factor (DLF), which is equal to $(1 + \mathrm{DLA})$, is defined based on the bridge span length. For moments in cantilevers and deck slabs, reactions, and shears, the DLF is given as a constant value of 1.30. For moments in simple or continuous spans, the DLF is specified as a function of the bridge span length as follows:

$$\mathrm{DLF} = \begin{pmatrix} 1.30 & L \leq 12\ \mathrm{m} \\ 1 + \dfrac{15}{L + 38} & L > 12\ \mathrm{m} \end{pmatrix} \tag{7}$$

where L = bridge span length (in meters) for positive moments and the average of adjacent span lengths for negative moments.

Australian Code

In the Austroads (2004) *AS 5100 Bridge Design Standard—Part 2: Design Load*, the DLA, which applies to both truck and uniformly distributed lane (UDL) loads, is given based on different load configurations, as shown in Table 5. The design loading configurations are not presented in detail here because of space limitations.

European Code

In *Eurocode 1: Actions on Structures—Part 2: Traffic Loads on Bridges* (CEN 2003), the DAF, which is based on a medium pavement quality and pneumatic vehicle suspension, has been included in road traffic load models. The built-in DAF is specified as a function of the bridge span length for one-, two-, and four-lane bridges. For one-lane bridges, the DAF for moment is specified as

$$\mathrm{DAF} = \begin{pmatrix} 1.7 & L \leq 5\ \mathrm{m} \\ 1.85 - 0.03L & 5\ \mathrm{m} < L < 15\ \mathrm{m} \\ 1.4 & L \geq 15\ \mathrm{m} \end{pmatrix} \tag{8}$$

and the DAF for shear is specified as

$$\mathrm{DAF} = \begin{pmatrix} 1.4 & L \leq 5\ \mathrm{m} \\ 1.45 - 0.01L & 5\ \mathrm{m} < L < 25\ \mathrm{m} \\ 1.2 & L \geq 25\ \mathrm{m} \end{pmatrix} \tag{9}$$

Table 4. DLA in OMT (1991)

Number of axles	DLA
1	0.4
2	0.3
3 or more	0.25

Table 5. DLA in Austroads (2004)

Traffic load configuration	Description	DLA
Wheel load, W80	Single wheel load of 80 kN	0.4
Axle load, A160	Two W80 wheel loads	0.4
Triaxle group, M1600	Combination of axle group and lane UDLs	0.35
Moving load, M1600	Combination of axle group and lane UDLs	0.3
Stationary load, S1600	Combination of axle group and lane UDLs	0
Heavy load platform load	Heavy load platforms	0.1

For two-lane bridges, the DAF for both moment and shear is specified as

$$\mathrm{DAF} = \begin{pmatrix} 1.3 - \dfrac{0.4}{100}L & L \leq 50\ \mathrm{m} \\ 1.1 & L > 50\ \mathrm{m} \end{pmatrix} \tag{10}$$

where L = bridge span length (in meters). For four-lane bridges, the DAF for both moment and shear is specified as a constant of 1.1. Under some unfavorable conditions, e.g., locations near expansion joints, an additional amplification factor $\Delta\phi$ needs to be considered

$$\Delta\phi = 1.3\left(1 - \frac{D}{26}\right); \quad \Delta\phi \geq 1 \tag{11}$$

where D = distance (in meters) from the location of the considered cross section to the expansion joint.

British Code

In BS 5400-2, *Steel, Concrete and Composite Bridges. Part 2: Specification for Loads* [British Standards Institute (BSI) 2006], two types of loading for highway bridges are considered, namely, the HA and HB loading for normal and abnormal traffic loads, respectively. For both loading types, an IM of 0.25 is included in the design load.

Japanese Code

The *Specifications for Highway Bridges* by the Japan Road Association (JRA 1996) defines the IM as a function of the bridge span length, as shown in Table 6. The expressions for the IM are similar to the format given in the AASHTO standard specifications. The IM for truck loading is specified as the same for all types of bridges while the IM for lane loading varies with different types of bridges.

Summary of Code Provisions

From a review of the different code provisions, it can be seen that (1) the specifications of the IM vary significantly between different national bridge codes, suggesting that there is no consensus for the evaluation of the IM in various countries; (2) a few common parameters are adopted in the expression of the IM by different design codes, including bridge span length (AASHTO, New Zealand, and Japanese codes), traffic load models (Australian, European, and British codes), and bridge natural frequency (Ontario and Chinese codes); and (3) the specifications for the IM are in simple formats despite the fact that the IM is influenced by many parameters.

Field Tests

General View

Field tests have proven to be the most reliable approach to investigate bridge dynamics under vehicular loads. From the 1950s to 1980s, many large-scale field tests were carried out in various

Table 6. Impact Coefficient in JRA (1996)

Bridge type	Loading type	IM
Steel	Truck and lane	$20/(50 + L)$
RC	Truck	$20/(50 + L)$
	Lane	$7/(20 + L)$
Prestressed concrete	Truck	$20/(50 + L)$
	Lane	$10/(25 + L)$

Note: L = span length (in meters).

countries. These works formed the basis of many national bridge design codes. A brief overview of these field tests is presented herein while a detailed review of these studies can be found in Paultre et al. (1992) and McLean and Marsh (1998). The past two decades has seen a significant increase in the installation of health monitoring instruments for long-span bridges; however, with the advances made in FEMs and the development of powerful simulation tools, large-scale field tests have been gradually replaced by numerical simulations. Fewer field tests of large scales have been reported in the past 2 decades.

In North America, large-scale field tests were conducted between the 1950s and 1980s to support the development of the AASHTO and Ontario codes. In 1958, the American Association of State Highway Officials (AASHO 1962) conducted field testing on 18 simply supported bridges with a uniform span of 15 m (50 ft). A maximum IM of 0.63 for displacements was observed while only 5% of measured values exceeded 0.40; the maximum IM for strains was 0.41 while only 5% exceeded 0.29. In addition, the IMs for strains were found to be smaller than those for displacements. Wright and Green (1963) reported the findings of a series of dynamic tests carried out on 52 highway bridges in Ontario from 1956 to 1957. A maximum IM of 0.75 was measured while most values were around 0.30. They also found that the IMs for bridges with measured fundamental frequencies from 2 to 5 Hz were relatively larger and that the IMs were significantly influenced by road surface conditions. From 1969 to 1971 a second series of field tests on 11 bridges in Ontario was performed by the OMTC (Campbell et al. 1971; Csagoly et al. 1972). It was found that the maximum IMs were between 0.3 and 0.85, which were obtained from bridges with a fundamental frequency range of 2 to 5 Hz. During the 1980s, a third series of dynamic testing in Ontario was implemented in support of the OHBDC (Billing 1984; Billing and Green 1984). The tests covered 27 highway bridges with different construction materials and various span lengths. They found that although the measured IMs exceeded 0.5 in some tests, the mean values of the IM (varying from 0.003 to 0.305) were comparatively moderate and the coefficients of variation of the IM (varying from 0.56 to 1.18) were relatively high.

Large scale field tests were also conducted in some other countries during almost the same period of time. Leonard (1974) and Page (1976) reported a series of field tests on 30 bridges conducted by the Transport and Road Research Laboratory in England. They observed that the measured IMs ranged from 0.09 to 0.75. During the 1970s, Shepherd and Aves (1973) and Wood and Shepherd (1979) presented the results of a series of field tests conducted on 14 bridges in New Zealand. The measured IMs ranged from 0.1 to 0.7. They also observed that the IM was strongly influenced by the dynamic characteristics of vehicles. Cantieni (1983, 1984) reported comprehensive dynamic load tests performed by the Swiss Federal Laboratories for Materials Testing and Research (EMPA) on 226 beam and slab-type highway bridges from 1958 to 1981. Most of these tested bridges were concrete structures. The results showed that the IM value could reach 0.7 for bridges with a fundamental frequency from 2 to 4 Hz.

Test Procedures and Data Processing

A static load test is usually carried out first to determine the unfavorable loading scenario and the maximum static response of the bridge to provide a reference for the calculation of the IM. The implementation of the static load test usually includes two common schemes. In the first method, a quasi-static test is carried out where the test vehicles are set to cross the bridge at a crawl speed. In the second method, the test vehicles are located at the longitudinal position that produces the maximum response of interest. The most unfavorable longitudinal position can be determined by using a simple method in which the bridge is modeled as a beam on which a series of concentrated forces corresponding to the vehicle axle weight are applied while the influence lines are used (Szurgott et al. 2011).

The dynamic load test can be divided into two types. In the first type, specific test vehicles travel through predetermined trajectories on bridges that have been closed to traffic. In the second type, the dynamic test is based on the measurement of real random traffic. Detailed testing procedures for dynamic testing of highway bridges can be found in Paultre et al. (1995). For the second type of dynamic load test, it is infeasible to conduct a static load test to acquire the corresponding static responses. In such a case, a low-pass digital filter can be adopted to derive the static responses from the measured dynamic responses in order to calculate the IM (e.g., Calçada et al. 2005a; Ashebo et al. 2007b). The criterion for the design of a low-pass filter is that the dynamic components of the response must be eliminated while the static response remains intact. To achieve this, the cutoff frequency must be less than the fundamental frequency of the bridge and greater than the frequency range that contains the static response.

Analytical Study

Although experimental studies remain the most dependable way to study the dynamic amplification of bridges under moving vehicles, the high cost and certain limitations have restricted their extensive applications. On the contrary, analytical study is an inexpensive method that is subject to fewer restrictions. With the fast development of computer technology, it has proven to be a very effective approach to solve VBI problems.

The earliest analytical study of moving loads can be found in Willis (1847), who investigated the case with a point mass moving on a simply supported massless beam at a constant speed. In subsequent studies, the vehicle was assumed as a single constant force moving across a simply supported beam (Kriloff 1905) and an analytical solution for the dynamic response of the beam was derived. Based on the previous studies, Timoshenko (1922) replaced the constant force with a harmonic force by considering the dynamic characteristic of vehicles. Inglis (1934) used a more realistic vehicle model, namely, a sprung mass with damping, to study the vehicle-induced vibration on railway bridges. Many of these early developments were summarized by Fryba (1972), who presented a comprehensive review and treatment on the solutions for moving load problems. Currently, with the advances in computer technologies and the emergence of commercialized finite-element (FE) packages, sophisticated three-dimensional (3D) bridge and vehicle models have been incorporated into relevant studies and numerical methods have been widely applied to obtain results that are in good agreement with those measured from field tests (e.g., Huang et al. 1993; Wang et al. 1994; Kwasniewski et al. 2006a, Shi 2006; Ashebo et al. 2007a).

Vehicle Model

The vehicle models adopted in previous studies can be classified into three types in terms of complexity: one-dimensional (1D) models (e.g., Chang and Lee 1994; Yang et al. 2004), in which the vehicle is modeled as a sprung mass of one or two degrees of freedom (DOFs); two-dimensional (2D) models (e.g., Chatterjee et al. 1994b; Chompooming and Yener 1995), in which a planar model with multiple axles is considered; and 3D complete vehicle models (e.g., Huang et al. 1992; OBrien et al. 2010). For example, a tractor-trailer 3D model was used in many studies to represent modern commercial trucks. Generally speaking, the vehicle body (tractor and trailer) was represented by a rigid mass with three DOF, i.e., vertical displacement, pitching rotation, and rolling rotation. Each vehicle axle was represented by a lumped mass with two DOFs, i.e., vertical displacement and rolling rotation. The tractor and trailer were interconnected through a pivot point (fifth wheel point) (Wang et al. 1992; Fafard et al. 1997). A typical three-axle vehicle model with 11 independent DOF is shown in Fig. 2. In addition to the mathematical models, Kwasniewski et al. (2006a) developed a FE truck model with 3D suspension systems and pneumatic and rotating wheels. The model was validated by experiments to accurately predict the truck dynamic response.

Bridge Model

In many early studies, a beam model was used to simulate the bridge superstructure. However, while this simple model gives some insight into the VBI problem it is unable to accurately reflect the spatial behavior of bridges. Consequently, spatial FE models for bridges were developed. The most common schemes adopted include plate models (e.g., Humar and Kashif 1995; Zhu and Law 2002; OBrien et al. 2010), grillage models (e.g., Huang et al. 1993; Wang et al. 1996b; Tan et al. 1998; Nassif et al. 2003; Ashebo et al. 2007a), and solid FE 3D models (e.g., Kwasniewski et al. 2006a, Deng and Cai 2010). A detailed review of bridge FE modeling can be found in González (2010).

Traffic Model

Because of the limitations associated with conducting long-term measurements of bridges under real traffic situations, many researchers have adopted the Monte Carlo method in order to simulate traffic flow (e.g., Zhang et al. 2001; González et al. 2008; Caprani et al. 2012; Caprani 2013). Usually, short-period weigh-in-motion data are first analyzed to identify the corresponding statistical distribution types for the representative parameters of traffic flow, including gross vehicle weight, vehicle speed, axle weight, axle space, headway, etc. Once the statistical characteristics of the traffic are known, the Monte Carlo method can be used to generate long-run traffic flows.

Road Profile Model

An accurate description of the road surface condition is very important in studying the VBI because road irregularity is the main excitation source of vehicle and bridge vibrations. Generally speaking, a road profile is represented by a zero-mean stationary random process and can be generated through an inverse Fourier transformation based on a power spectral density (PSD) function (Dodds and Robson 1973; Honda et al. 1982).

Recently, more realistic road profile models have been developed by some researchers. One important improvement is the introduction of the 2D road profile model. In many previous studies, the road profile is assumed to be identical throughout the bridge transverse direction. However, this assumption may be significantly different from in situ measurements, and in such a case the rolling action of the vehicle cannot be captured. For this reason, Liu et al. (2001, 2002) modeled the longitudinal road profiles as correlated random processes along the deck transverse direction. They concluded that a better correlated road profile produces larger IMs depending on the level of correlation between the road profiles in the longitudinal direction. Based on their studies, the application of two identical (i.e., fully correlated) road profiles is more conservative but still acceptable in practice. Oliva et al. (2013) developed an efficient method to generate pairs of parallel profiles. They found that by neglecting the difference between the two parallel longitudinal profiles, the dynamic response of the bridge and the vertical and pitching accelerations of the vehicles tend to be overestimated; as a result, larger DAFs will be produced, which is similar to the findings by Liu et al. (2001, 2002).

Solution of Vehicle-Bridge System

Generally speaking, two common approaches have been used to solve VBI systems, i.e., the iterative method (e.g., Wang and Huang 1992; Chatterjee et al. 1994a; Broquet et al. 2004) and the coupled method (e.g., Kim et al. 2005; Henchi et al. 1998; Deng and Cai 2010). In the first approach, two sets of equations for the vehicle and bridge are solved separately through an iterative procedure by using the displacement compatibility and force equilibrium conditions at the vehicle-bridge interface. However, in the coupled method a coupled equation is formulated by assembling the two sets of equations, which results in a single matrix for the mass, damping, and stiffness of vehicle-bridge system. Solving the equation is usually achieved by using direct integration methods such as the Newmark method (e.g., Fafard et al. 1998; Broquet et al. 2004) and the Runge-Kutta method (e.g., Wang et al. 1992; Cai et al. 2007). Although accurate results can be derived from the first method, a large amount of calculation effort may be required to achieve convergence. Furthermore, modal superposition is a general technique used in solving VBI problems because it significantly reduces the calculation effort by reducing the size of the matrices in the equation.

In addition, Yang and Lin (1995) and Yang and Wu (2001) proposed VBI elements that consist of both bridge elements and the suspension units of the vehicle that are directly in contact with the

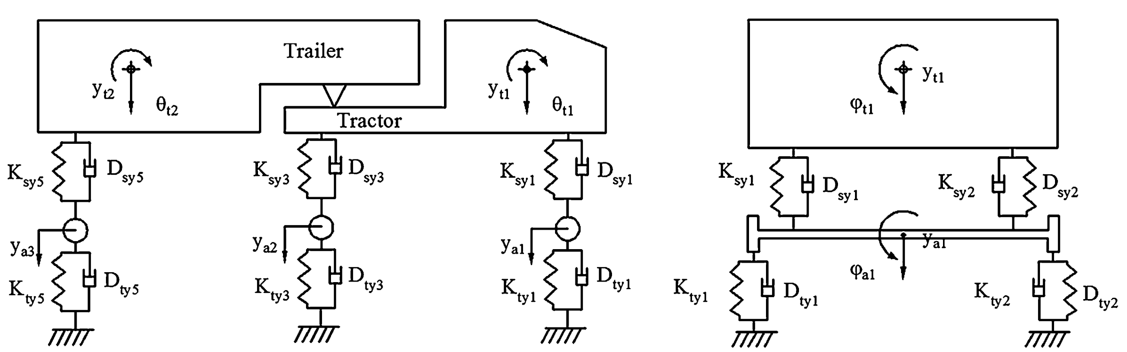

Fig. 2. Vehicle Model HS20-44 (data from Huang et al. 1992)

bridge elements. By using the modified dynamic condensation method (Paz 1989), all DOF of the vehicle are then condensed into the developed element. In this case, the conventional FEM can be applied to assemble the equations, leading to more efficient computation.

Parametric Study of the IM

Road Surface Condition

Road surface roughness has been identified as a major source of excitation in vehicle-induced bridge vibrations. Numerous studies have shown that the IM increases significantly with the deterioration of the road surface (e.g., Wang and Huang 1992; Huang et al. 1995a, Broquet et al. 2004; Kim et al. 2007; Deng and Cai 2010). Besides, many studies have also indicated that poor road surface condition is an important cause in the underestimation of the IM in bridge codes (Wang et al. 1994; Liu et al. 2001; Zhang et al. 2006; Ding et al. 2009; Deng and Cai 2010). In light of this, regular maintenance of the road surface condition is believed to be a very cost-effective approach that reduces the dynamic impact of moving vehicles in bridge management.

Furthermore, some studies have focused on the investigation of the correlation between the IM and the road surface condition. Park et al. (2005) investigated the influence of road roughness on the IM based on a dynamic test of 25 highway bridges in Korea. The results from regression and correlation analyses showed that the IM increases almost linearly with the international roughness index (IRI) or roughness coefficient. However, Li et al. (2006) showed that neither the IRI nor PSD provides accurate measure of the extent to which the road roughness affects the dynamic amplification. They proposed the dynamic amplification estimator (DAE) as the indicator of the DAF and found that the DAE provides an accurate estimate for the DAF under good surface conditions. OBrien et al. (2006) pointed out that the DAE is vehicle specific and proposed the bridge roughness index (BRI), which is independent of vehicle properties, as an indicator of the DAF. They found a good correlation between the BRI and DAF. Nevertheless, the BRI is still specific to vehicle fleet properties, bridge span lengths, and load effects. In fact, OBrien et al. (2006) believed that there is rarely a simple road roughness measure that is both universally applicable and well correlated with the DAF as a result of the considerable variations of many other influencing parameters, e.g., vehicle properties, traffic flow characteristics, bridge span lengths, and load effects.

In addition to the road surface roughness, the approach span condition also has a strong influence on the IM. Cai et al. (2007) found that large initial oscillations of vehicles induced by faulting at expansion joints and the ends of approach slabs could lead to considerably high IMs that may exceed the values specified in the AASHTO codes. Similar observations were also made by Shi et al. (2008), Moghimi and Ronagh (2008), and González et al. (2011). This phenomenon should be noted in bridge construction and maintenance. To reduce or slow down the differential settlement between the bridge abutment and the embankment soil, and thus the faulting at the expansion joints, some measures can be taken, such as increasing the compactness of the embankment soil.

Bridge Span Length and Fundamental Frequency

The natural frequency of a bridge is usually related to its span length. In previous studies (Cantieni 1983; Billing and Green 1984; Memory et al. 1995), many empirical formulas were proposed to describe the relationship between the bridge fundamental frequency and the span length. Although these formulas vary in mathematical form, they consistently imply that the bridge fundamental frequency decreases as the span length increases.

As previously presented, some bridge design codes specified the IM as a simple function of the bridge span length. However, based on previous studies, it seems that the relationship between the IM and bridge span length is somewhat unclear. Field tests conducted by EMPA showed a poor correlation between the IM and bridge span length (Cantieni 1983). Coussy et al. (1989) concluded that the IM is not a function of the bridge span length. Huang et al. (1993) observed that the IMs at the quarter-span and midspan have different variation trends with respect to the bridge span length. Chang and Lee (1994) concluded that the IM does not vary significantly with the bridge span length. Schwarz and Laman (2001) found high coefficients of variation for the DLAs of each bridge span and thus concluded that there is no definable relationship between the DLA and bridge span length. Li (2005) concluded that although the IM tends to decrease with the increase of the bridge span length, this is not always the case because the IM will be amplified significantly as a result of the resonance of the vehicle-bridge system when the fundamental frequency of bridges approaches that of the vehicle. This phenomenon—i.e., the matching of vehicle and bridge frequency—leads to significantly larger IMs, and has also been indicated in many studies (e.g., Huang et al. 1992; Green et al. 1995; Schwarz and Laman 2001; Li et al. 2008; Ding et al. 2009). However, Yang et al. (1995) concluded that the influence of the vehicle-bridge frequency ratio on the IM is insignificant because the variation of the IM with the frequency ratio is small. Pan and Li (2002) discovered that the maximum dynamic response does not occur in the case when the vehicle-bridge frequency ratio is equal to 1. They believed that the excitation frequency in the vehicle-bridge system is a combination of many factors, including vehicle speed, road roughness, and bridge frequency, instead of the vehicle frequency alone.

Bridge Type

In the past, the IMs of different bridge types have been studied extensively, both experimentally and analytically. As discussed in the previous section, during the time period between the 1950s and 1980s, large-scale field tests were conducted on various bridge types in different countries. In the past two decades a series of numerical studies on the IM of a large variety of bridge types, including cable-stayed bridges (Wang and Huang 1992; Huang and Wang 1992); steel and concrete girder bridges, both curved and straight (Huang et al. 1992, 1993, 1995a, b, 1998; Wang et al. 1992, 1993, 1996a, b; Huang 2001, 2008); and arch bridges (Huang 2005, 2012) have been conducted. Nevertheless, a significant amount of research efforts in the literature has been focused on short to medium–span girder bridges because these bridges account for a large proportion of highway bridges worldwide. In contrast, the amount of research on the IM of long span bridges, including suspension bridges and cable-stayed bridges, has been significantly less because it is generally believed the IM of long span bridges is relatively small and could even be neglected compared with the large dead load effect. It should be noted that it is not the aim of this paper to summarize all findings on IMs of all bridge types. Therefore, considering space limitations, only a few typical bridge types with a substantial amount of information available in the literature are summarized here. Furthermore, findings related to the influence of other parameters regarding different bridge types are not repeated in this section.

Studies on I-girder bridges have suggested that (1) there exists a considerable variation of the IM for different girders because girders that carry larger static loads usually have smaller IMs (Huang et al. 1992, 1993; Wang et al. 1992; Kim and Nowak 1997; Schwarz and

Laman 2001); (2) with the increase of girder spacing, the IMs for interior girders decrease while the IMs for exterior girders do not change significantly (Huang et al. 1992, 1993); and (3) the total number of longitudinal girders also affects the IM, and this effect may have some correlation with the road surface condition (Wang et al. 1996b). In addition, the dynamic behavior of horizontally curved girder bridges may be different from that of straight bridges. In the AASHTO codes, curved bridges have been previously designed using separate codes, e.g., the AASHTO (2003b) *Guide Specifications for Horizontally Curved Highway Bridges*, while the current AASHTO LRFD code has unified the provisions for straight and curved girder bridges. Most of previous studies have focused on the IM of horizontally curved box-girder bridges and they indicated that (1) the IM is insensitive to curvature for radii greater than 1,219.2 m (4,000 ft) and is distinctly influenced by the curvature for radii equal to or less than 243.84 m (800 ft) (Schelling et al. 1992); (2) the IMs of the vertical bending moment and normal stress for curved box-girder bridges are smaller than those for corresponding straight bridges (Huang et al. 1998; Huang 2001); (3) the span-to-radius ratio has a notable effect on the IM (Samaan et al. 2007); and (4) the IM appears to decrease with the decrease of radius and the increase of span length (Huang 2001).

The amount of studies on the IMs of long span bridges under vehicular loads is significantly less compared with the amount of earthquake- and wind-related studies. Based on numerical simulations of cable-stayed bridges under vehicle loads, Wang and Huang (1992) and Huang and Wang (1992) concluded that (1) the suspended center span of cable-stayed bridges has a strong influence on the IM because the existence of a hinge at the midspan causes a considerable increase of the IM; and (2) the influence of the arrangement of cables on the IM is insignificant. Moreover, their numerical simulation results indicated that IM values greater than 0.6 were observed at some locations of the bridge, even under good road surface conditions. However, field test results on a cable-stayed bridge in Portugal provided by Calçada et al. (2005a) showed that the maximum IMs are basically less than 0.2. Although these results may not be comparable, this significant difference can still serve as a reminder for engineers in practice that field test results are still the most dependable data to be relied upon in cases where large discrepancies exist. In addition, previous studies on arch bridges have shown that (1) the rise-to-span ratio is the most important geometric parameter and the IMs of different bridge responses have different variation trends with the rise-to-span ratio (Huang 2005); and (2) the difference between IMs for fixed and two-hinge arch bridges is insignificant (Huang 2012).

It should be stressed that even for bridges of the same type, various parameters such as the road surface condition, load effect, vehicle type, reference location, structural member, etc., could also lead to significant variations of the IM. For different types of bridges, some of these parameters may have similar effects on the IM while others may not. Therefore, it is difficult to draw a meaningful comparison of IMs between different bridge types from the available results.

Bridge Material and Damping

Although few studies have been conducted to provide comprehensive examinations of the effect of various bridge materials on the IM, some researchers believe that bridge materials do not have a significant influence on the IM (Paultre et al. 1992; McLean and Marsh 1998). In bridge designs, some design codes distinguish the IM between bridges with various construction materials. For example, the Chinese general code (MTPRC 1989) and Japanese specifications (JRA 1996) provide different impact functions for steel and concrete bridges. In addition, the *Canadian Highway Bridge Design Code* (CSA 2006) specifies that the DLA for wood components shall be multiplied by a reduction factor of 0.7. In the AASHTO (2012) LRFD code, the DLA is not considered for wood components because of the high material damping of wood.

In addition, fiber-reinforced polymer (FRP) materials have been gradually introduced into the construction of new bridges and retrofitting of existing bridges because of their high strength, light weight, and good corrosion resistance. The dynamic characteristics of FRP bridges may be different from bridges constructed using conventional materials because of the differences in material weight and stiffness. Zhang et al. (2006) compared the vehicle-induced dynamic responses of a FRP slab bridge and a concrete slab bridge and found that the IM of the FRP bridge is much smaller than that of the concrete bridge, which was also reported by Hag-Elsafi et al. (2012). Of interest is the fact that the damping ratios of FRP bridges were found to be much lower than those of concrete bridges (Aluri et al. 2005). In such a case, the IM of FRP bridges would be expected to be higher than that of concrete bridges because many previous studies have suggested that the IM decreases with the increase of bridge damping (e.g., Huang and Wang 1992; Wang et al. 1994; Huang et al. 1995a, b; Azimi et al. 2011).

Vehicle Speed

Vehicle speed has been regarded as an important parameter influencing the IM. Chang and Lee (1994) concluded that the IM increases with the increase of vehicle speed. This tendency was also found in many studies (e.g., Sennah et al. 2004; Calçada et al. 2005b; Kwasniewski et al. 2006b; Li et al. 2008; Ding et al. 2009). However, Laman et al. (1999) found no clear relationship between the vehicle speed and IM, which was also reported by some researchers (e.g., Green et al. 1995; Broquet et al. 2004; Deng and Cai 2010; Ashebo et al. 2007b; Azimi et al. 2011). In fact, the influence of vehicle speed on the IM is associated with many other factors, as can be seen from many studies. For example, Hwang and Nowak (1991) found that the variation of the IM with vehicle speed is related to the vehicle weight; Huang et al. (1992) observed that the corresponding speeds for the maximum IM vary with different road surface conditions, span lengths, and reference locations; Huang et al. (1995a) found that the IMs of interior and exterior girders vary differently with vehicle speed; Huang (2005) found that the IMs of different load effects have different tendencies of variation with vehicle speed; and Huang (2012) observed that fixed and two-hinge arch bridges have different variations of the IM with vehicle speed.

Moreover, many researchers (e.g., Wang et al. 1992; Green and Cebon 1997; Yang et al. 2004) considered the vehicle speed in terms of a dimensionless speed parameter defined as

$$S = \frac{\pi v}{\omega_b L} \tag{12}$$

where v = vehicle speed; ω_b = fundamental circular frequency of a bridge; and L = bridge span length. This parameter can be interpreted as the ratio of the loading frequency to the fundamental frequency of the bridge. The loading frequency, namely, $\pi v/L$, corresponds to the frequency of the forced vibration of a simply supported beam subject to a constant force moving at a constant speed, namely, no interaction is assumed between the vehicle and bridge. Smith (1988) defined an IM as $1/(1-S)$ for a simply supported beam under a moving constant load. According to this expression, the IM increases with the increase of vehicle speed because the loading frequency is usually small compared with the fundamental frequency of bridges as a result of the speed limit.

Yang et al. (1995) examined the IMs for simple and continuous beams under moving vehicles and found that the IMs at the midspan are linearly proportional to the speed parameter. However, Chatterjee et al. (1994b) found no particular pattern for the variation of the DAF with the speed parameter, based on the analysis of the vibration of suspension bridges under vehicular loads.

It can be seen that the issue concerning the influence of vehicle speed on the IM is a controversial one because previous studies have yielded inconsistent findings. This may be partly a result of the various models and methods adopted in various studies. Nevertheless, the inconclusive relationship between the vehicle speed and IM indicates that this influencing mechanism is quite complicated.

Furthermore, many studies have focused on determining the critical speed at which the maximum IM occurs. Shi et al. (2008) used the following equation to predict the critical speed:

$$\frac{v}{L_v} n = f \quad (n = 1, 2, 3, \ldots) \tag{13}$$

where v = vehicle speed; L_v = axle spacing of the vehicle; and f = fundamental frequency of the bridge. The basis for this equation is that the resonance of the vehicle-bridge system will occur when the excitation frequency of the moving load becomes equal to the fundamental frequency of the bridge. In the Shi et al. (2008) study, the prediction of critical velocities using Eq. (1) agreed well with those obtained from the numerical study. However, this method is only applicable when the bridge span length is relatively short compared with the axle spacing and the vehicle has multiple axles with similar spacing. Brady et al. (2006) studied the case of a single point load crossing a simply supported beam and concluded that a set of critical frequency ratios exists. These critical ratios correspond to a series of critical speeds where the maximum DAFs occur, and these critical speeds were in good agreement with those obtained from the FE analysis. In a companion study (Brady and O'Brien 2006), the event of two vehicles crossing was examined using a similar method. González et al. (2010) extended the single point load model to a series of point loads in accordance with the axle spacing and weight of vehicles with typical configurations. Similarly, the critical speed was found for critical meeting events of two trucks traveling in opposite directions. The prediction using a point load model was validated using numerical examples and field test results.

It is undeniable that these studies have given some insight into the mechanism in which the maximum IM takes place. However, the prediction of critical speeds using simple models is still subject to some limitations. For example, the point load model will be less accurate when the mass ratio of the vehicle to the bridge becomes large. In reality, the speed of vehicle, as a random parameter, varies from one vehicle to another and from time to time, suggesting that it is difficult to account for the effect of speed on the IM in bridge design and assessment. Nevertheless, the prediction of critical speed may be potentially useful in determining the permitting speed in bridge management.

In addition, while many studies have considered vehicle speed as a constant during the VBI, changes in speed, i.e., deceleration (braking) and acceleration, are often encountered in practice. With the braking of the vehicle, a pitching moment is produced and the contact forces between the wheels and the bridge change significantly, which may increase the bridge vibration and lead to higher IMs. Previously, some researchers focused on the development of methodologies to consider vehicle braking and acceleration (Yang and Wu 2001; Ju and Lin 2007; Azimi et al. 2013). Other researchers found that vehicle braking may significantly amplify the IM (Gupta and Traill-Nash 1980). Law and Zhu (2005) observed very large IMs for short braking rise times. This was also reported by Yin et al. (2010), who found the braking rise time and braking position to be two important parameters affecting the maximum IM.

Vehicle Weight

The influence of vehicle weight on the IM has been investigated by many researchers. It is believed that the IM tends to decrease as the vehicle weight increases (e.g., Huang et al. 1993; Broquet et al. 2004; Ashebo et al. 2007b). This tendency is consistent with the finding that the IM decreases with the increase of the static load effect as indicated in many studies (e.g., Kim and Nowak 1997; Laman et al. 1999; Schwarz and Laman 2001; Huang 2012).

Hwang and Nowak (1991) observed that the dynamic load effect does not vary significantly with the increase of vehicle weight while the static load effect increases significantly. In addition, they found that the correlation between the dynamic and static load effects is very weak or even nonexistent, as was found by Moghimi and Ronagh (2008). However, Broquet et al. (2004) found that both the dynamic and static effects increase as the vehicle mass increases, which were also reported by Chan and O'Connor (1990). Nevertheless, in all cases, the IMs decrease with the increase of the vehicle weight. Although light vehicles are more likely to produce large IMs, these large IMs are often of little practical significance as a result of the corresponding small static load effects (Kwasniewski et al. 2006b).

Number of Axles

In the OHBDC (OMT 1991) and CHBDC (CSA 2006) codes, the DLA is specified as a function of the number of axles. However, Laman et al. (1999) observed a very wide range of DLAs for vehicles with the same number of axles. Schwarz and Laman (2001) found no statistical relationship between the DLA and the number of vehicle axles. Ashebo et al. (2007b) also concluded that the IM is nearly independent of the number of axles. It is important to note that the data used in these studies were measured from field tests under normal traffic conditions. Thus, even for vehicles that have the same number of axles, they may still vary in vehicle weight, axle spacing, suspension characteristic, etc. These variables are very likely to cause the variations of the IM induced by vehicles with the same number of axles.

Number of Vehicles

Many studies have shown that the IM for multivehicle presence is lower than that for a single vehicle loading (e.g., Hwang and Nowak 1991; Wang et al. 1992; Humar and Kashif 1995; Ashebo et al. 2007b). This is because the presence of multiple vehicles increases the total static load effect and thus reduces the IM. In real life, the probability of single vehicle presence is relatively low, which implies that the IM obtained from studies that considered only a single vehicle loading may be conservative to some extent. Therefore, in some design codes, e.g., the AASHTO (1994) LRFD code, a certain reduction in the DLA along with the LL is allowed for when multiple vehicles are present. However, in addition, it is interesting to note that some researchers have found that the IMs obtained from symmetrical loading of two vehicles traveling abreast of each other are close to those obtained from a single vehicle loading (Kashif 1992; Deng and Cai 2010). This may be attributed to the synchronized interaction forces of the two vehicles. For two vehicles traveling in parallel lanes, where one lags behind the other, the IMs were found to be much smaller than those for two vehicles traveling side by side (Humar and Kashif 1995).

Vehicle Suspension Type

The dynamic characteristics of the vehicle depend largely on the design parameters of the vehicle suspension, including damping, stiffness, and vehicle natural frequencies. Green et al. (1995) examined the effect of vehicle suspension on the bridge dynamics and concluded that the IM induced by air-sprung vehicles is lower than that by leaf-sprung vehicles, as was found by Green (1997), Varadarajan (1996), Cantieni and Heywood (1997), and Kirkegaard et al. (1997). This is because air-sprung vehicles exert smaller dynamic loads on bridges and they have more viscous damping. Kirkegaard et al. (1997) found that lower suspension stiffness results in a smaller DAF while the change of damping does not have a significant effect on the DAF. Kwasniewski et al. (2006b) concluded that the high IMs observed in the field test results were caused by very stiff suspension of the truck. Szurgott et al. (2011) also found high DLAs for a truck with stiff vehicle suspension. They believed that modern vehicle suspension systems with efficient springs and dampers can serve as an effective tool to mitigate vehicle and bridge vibrations and can thus reduce the DLA. However, Yang et al. (1995) found that the effect of suspension stiffness on the IM is insignificant.

Ideally, the vehicle suspension is carefully designed such that the natural frequencies of the vehicle do not coincide with the common frequency range of highway bridges. However, this may be difficult to achieve in practice. Thus, the use of soft and well-damped vehicle suspension is suggested to reduce the IM (Green et al. 1995). In addition, Harris et al. (2007) proposed an innovative approach to diminish the dynamic amplification of bridges due to heavy vehicles by using a real-time intelligent control system that is capable of adjusting the damping coefficient of the suspension system to an optimum value for the vehicle prior to crossing the bridge. The approach was validated by a numerical model and was found to achieve desirable performance especially for rough road profiles.

Vehicle Loading Position

Vehicle loading position determines the distribution of wheel loads on the bridge as well as the vibration modes displayed by the bridge, which in turn influences the dynamic response of the bridge. Huang et al. (1993) found that the IMs of different girders of multigirder concrete bridges vary significantly with different vehicle loading positions. They concluded that the IM decreases with the increasing static wheel-load distribution factor. However, this relationship is not necessarily accurate for asymmetrical loading positions because of the effect of torsion. Similar observations were also made in many other studies (e.g., Wang et al. 1994; Kirkegaard et al. 1997; Kwasniewski et al. 2006b; Moghimi and Ronagh 2008). Generally speaking, a higher static load effect leads to a lower IM, as discussed previously. From the design perspective, large IMs are not necessarily relevant, thus the IM should be calculated based on the loading positions that produce the maximum static load effects.

Concluding Remarks

In this paper, a comprehensive review of the state-of-the-art research on the subject of the dynamic IM of highway bridges is presented. On the basis of recent analytical and experimental findings, the following conclusions can be drawn:

1. There exists a significant variation in bridge code provisions for the dynamic IM. The main influencing parameters of the IM adopted in design codes include the bridge span length, fundamental frequency, and number of vehicle axles. Many previous studies showed that the relationships between these parameters and the IM are somewhat indeterminate. Furthermore, the IMs obtained for different load effects may be significantly different, suggesting that it may be unreasonable to assign a constant value of the IM for all types of load effects (such as displacement and moment) in the practice of bridge design and assessment. Thus, it is recommended that the IMs be specified based on different load effects in the design practice.
2. Road surface roughness is a major factor affecting the IM. The IM increases significantly with the deterioration of road surface roughness. Thus, regular maintenance of the road surface provides a very effective means of reducing the IM of existing bridges. In addition, attention should also be given to the condition of approach spans and deck joints.
3. It seems that there is not enough information to draw conclusions regarding the influence of bridge materials on the IM. Furthermore, based on the available information of IMs for different types of bridges, it is difficult to determine the influence of bridge type on the IM because of the influence of many other factors.
4. Vehicle speed is an important influencing parameter of the IM. However, the relationship between the vehicle speed and IM seems inconclusive, implying that this influencing mechanism is complicated. Moreover, vehicle suspension characteristics also have a significant influence on the IM. The design of vehicle suspension is critical for the mitigation of vehicle-induced vibration.
5. Generally speaking, the IM decreases as the vehicle weight increases, which is a result of the fact that higher static load effects generally result in smaller IMs. Similarly, higher wheel load–distribution factors also lead to smaller IMs. These facts suggest that unfavorable vehicle loading positions should be used when evaluating the IM for a specific bridge to avoid excessively large IMs without practical significance.

Based on a review of the main findings and advances in this field, the following problems have been identified and corresponding suggestions are tentatively made regarding the future investigation of the dynamic IM:

1. The LL is generally well defined in various national bridge codes. However, the IMs specified in bridge codes are not necessarily consistent with the definitions of LLs. For example, in the AASHTO LRFD code, the IM is applied to truck load only and is not considered for lane load. The rational for this combination of the LL and DLA has been questioned by some researchers. Thus, further research is required to verify whether the total LL effect under such a combination could accurately reflect the maximum possible total effect induced by vehicular loads. Moreover, the basis of the current AASHTO LRFD provision is the numerical studies conducted by Nowak et al. (1990) and Hwang and Nowak (1991) in which a single truck and two-truck loading events were adopted in the simulations. The extent to which the dynamic load effect caused by this simplified loading model agrees with that caused by real traffic loading needs to be further investigated.
2. The difference in the impact behavior of various bridge types needs further research. Currently, most bridge codes do not distinguish between IMs for different bridge types. Moreover, various cross-section configurations and curvatures of bridges could also lead to different IMs. Therefore, the IMs of various bridge types should be treated differently based on practical experiences in the design and evaluation of bridges if no better information is available.
3. Construction materials may have a significant influence on the IM. Nowadays, with the introduction of light weight and high-strength materials, e.g., FRP, into the bridge industry, the difference between IMs for conventional materials and new materials has raised more concerns. Thus, more extensive

investigation, especially based on field measurements, concerning the IM of various materials is needed.

4. A uniform standard for field test procedures and data processing methods needs to be developed to obtain more reliable and comparable data from field tests. Such a standard should include instructions for the installation location of instruments, types of transducers, vehicle loading scenarios, date collecting and processing approaches, etc.
5. No consensus has been reached on the relationship between the IM and vehicle speed, probably because the influence of velocity on the IM may correlate with those of many other factors. Further in-depth studies are required to reveal the complex relationship between the IM and vehicle speed. In addition, it is worth mentioning that significant progress has been achieved in relation to this problem for railway bridges (Yang et al. 1997; Yau et al. 1999; Savin 2001; Hamidi and Danshjoo 2010). Therefore, this IM research conducted for railway bridges could serve as a reference for the study of highway bridges.
6. In the bridge codes the IMs are usually intended for the design of new bridges. However, when evaluating in-service bridges, especially for short bridges with poor road surface conditions, adopting the IMs in bridge design codes may lead to unsafe assessment results for the load-carrying capacity of bridges (Deng et al. 2011). In such cases, reasonable IMs based on practical road surface conditions should be considered. Whenever feasible, the full advantage of conducting field testing should be taken because it is still the most reliable approach used to obtain accurate IMs.
7. Specific instructions for the application range of the IMs in bridge codes are still lacking and more research in corresponding aspects is required. For example, because IMs are traditionally obtained from global load effects, such as the displacement or bending moment at the bridge midspan, it may be irrational to apply these IMs to the design of local structural components, e.g., the deck slab. As another example, although vehicle braking may produce significantly larger IMs, this phenomenon is not emphasized in many bridge design codes. Instead, only the influence of the longitudinal force induced by vehicle braking on bridges is considered.
8. The IM is affected by a large number of variables, and the level of influence of some of these variables is still unclear. Thus, a sensitivity study is needed in the future. Some recently developed methods for handling structural uncertainties—for instance, the interval analysis method—can provide promising tools to investigate the sensitivity of the IM to the influencing parameters (Liu et al. 2013). Unlike the traditional probabilistic methods, which require a large amount of measurement data to determine the statistical characteristics of the uncertain parameters, the interval analysis method defines variables in close bounded intervals that can be obtained based on limited data. This is a significant advantage because field measurement data, still being the most reliable data, are usually expensive and difficult to obtain. Furthermore, to accurately evaluate the dynamic amplification, reliability-based methods are recommended in future research. Recently developed hybrid reliability approaches (Jiang et al. 2011, 2012), which integrate the traditional probability approach and nonprobability interval analysis, may provide useful tools for the reliability analysis of the IM.

Acknowledgments

The authors gratefully acknowledge the financial support provided by the National Natural Science Foundation of China (Grant No. 51208189) and the Excellent Youth Foundation of Hunan Scientific Committee (Grant No. 14JJ1014). The constructive comments from the editor and reviewers are also greatly appreciated.

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