# State of the Art Review

Structural analysis of traffic loading on bridges is performed primarily for the purposes of design or evaluation. The methods of analysis and technological tools historically available have limited the analyses to evaluate the traffic as static loads (i.e. location and magnitude of load remain constant with time), rather than as moving masses where the true response of the structure is actually due to the interaction of the two dynamic systems. The industry accounts for this disparity by applying a “dynamic amplification” factor to the static load or response, thereby attempting to increase the calculated response to match that which the bridge may actually experience under moving vehicles.

The magnitude of this factor (IM or DAF) differs depending on the reference code being used, but in many cases, has been developed from extensive surveying of existing structures and the amplifications they experience.

## Dynamic Amplification in Bridge Codes

The various design and evaluation codes handle dynamic amplification in slightly different ways, but they all specify a factor that is to be applied to the loading model. In the current AASHTO (2014) *LRFD Bridge Design Specification* the impact factor is dependent on road surface roughness, ranging from 0.1 to 0.3. The AASHTO (2018) *Manual for Bridge Evaluation* specifies an impact factor of 0.33 (0.15 for fatigue limit states, 0.75 for joints). The British code includes an IM of 0.25 in its two design load models.

The factors specified by the Ontario and Canadian Highway design codes are dependent on the number of vehicle axles, with lower factors for more axles. A factor of 0.25 is specified for vehicles with 3 or more axles. Similarly, the Australian code specifies factors based on load type. An impact factor of 0.4 is specified for wheel and axle loads, and 0.35 for triaxle truck and lane load. (Canadian Standards Association, 2006)

The Chinese code’s factors are a function of bridge’s first natural frequency, with a lower bound of 0.05 and an upper bound of 0.45. A bridge with a first natural frequency below 5 Hz would have a specified impact factor less than 0.25. (Ministry of Transport of the People’s Republic of China (MTPRC), 2004)

The New Zealand, European, and Japanese codes all specify factors as a function of bridge span length with the factors decreasing with increased span length. The New Zealand code has a maximum IM of 0.30 for span lengths less than 12 meters (New Zealand Transport Agency (NZTA), 2013). The equation for IM specified by the Japanese code is different for different bridge types (Japan Road Association (JRA), 1996). A steel or RC bridge with a span length of 40 feet for instance would be assigned an IM equal to 0.32.

The equation specified by the European code differs dependent on the number of lanes. For a single-lane bridge, the impact factor may range from 0.4 to 0.7, while that for a two-lane bridge ranges from 0.1 to 0.3. (European Committee for Standardization (CEN), 2003)

## Experimental Evaluation of Amplification Factors

A large number of field tests were carried out in various countries over the years, in-part, for the development of bridge design codes. In the 1950’s, AASHTO sponsored a major investigation, for which 18 newly constructed bridges were selected for the purpose of testing and determining the dynamic effects of moving vehicles on the bridges. This study concluded that the dynamic amplification generally increases with increased vehicle speed, is sensitive to vehicle suspension performance, and that the initial oscillations of the vehicle are responsible for a large amount of uncertainty in the dynamic response of the bridge. The maximum dynamic amplification factor recorded from these tests was 1.63 for displacements and 1.41 for strains, however, 95% of the measured amplification factors fell below the value specified by the current code at the time. (Paultre et al., 1992)

In 1956 and 1957, field tests were completed on 52 bridges in Canada, with specific attention paid to dynamic amplification. Amplification factors were observed as high as 1.75, while values near 1.30 were more typical. These tests concluded that the amplification factors were higher for flexible bridges and were greatly affected by the road surface roughness and irregularities on the bridge as well as approach (Wright and Green, 1964). In fact, Write and Green recommended that irregularities in the road profile should be eliminated, as this was more influential on dynamic amplification than the many other structural parameters considered (Paultre et al., 1992). Additional tests were carried out in Canada in the 1970s and 1980s. Amplifications factors were obtained as high as 1.85 with higher factors obtained from bridges with first fundamental frequencies of 2 to 5 Hz (Billing, 1984). Similar results were obtained from field tests of more than 200 bridges in Switzerland, with amplification factors as high as 1.7 for bridges with a first fundamental frequency between 2 and 4 Hz (Cantieni, 1983). (Deng et al., 2015)

A more recent field test was completed in Florida of a 3-span prestressed multi-girder bridge. Amplification factors were determined experimentally by measuring the bridge response from loaded five-axle trucks. The amplification factors for one truck was found to be 1.82 while that for two trucks was found to be 1.50 and were observed at higher speeds (80 km/h). The authors were able to reasonably reproduce the bridge response recorded in the field by using FE simulation and incorporating the measured road profile of the approach. By comparing responses with and without road profile, the authors were able to show that road surface irregularities have a significant impact on dynamic amplification. (Kwasniewski et al., 2006b)

## Modeling Vehicle-Bridge Interaction

While much can be gleaned from experimental investigations, they are often expensive, and require a large amount of resources and time. For this reason, analytical methods are often employed, especially where a large number of situations need to be investigated (i.e. parametric studies).

### Model Forms

In early studies, vehicle-bridge interaction was analyzed by representing the bridge as a beam, and the vehicles as point forces or point masses moving across the beam (Willis, 1849). As computational tools advanced, bridge models gained complexity and are now commonly modeled as grillage models (Ashebo et al., 2007) or full 3D FE models (Kwasniewski et al., 2006a). Similarly, the vehicle models have advanced to multiple degrees of freedom (DOF). The minimum vehicle model is composed of a single sprung-mass system (Yang et al., 2004). It often consists of a single DOF but may have two if the wheel weight and tire stiffness is to be modeled separately from the suspension system. Two dimensional models can also be used, whereby each axle is represented by a sprung-mass system that may optionally be connected (Chatterjee et al., 1994). Finally, a full 3D model of the vehicle may be constructed, for which the vehicle body is modeled as a rigid mass (or multiple connected masses) with multiple DOF, and the suspension system will be modeled as lumped masses connected to the vehicle body and bridge surface via spring-dashpots elements (OBrien et al., 2010).

### Traffic Models

In some cases, the effect of traffic is considered, and multiple vehicle models are employed in the analysis. The traffic parameters (e.g. weights, spacing, speed, etc.) can be obtained directly from weigh-in-motion (WIM) data but are often generated using statistical methods, whereby the distributions of traffic parameters are derived from WIM data or from traffic flow simulations. The traffic model is then formed using Monte Carlo methods. (Caprani, 2012; Zhang et al., 2001)

### Profile Generation

The interface between the traffic/vehicles and structure is the road surface. The motion of the vehicle tires over the bridge is dictated by the geometry of that surface (i.e. road profile). Therefore, it is important that it be included in any modeling effort. The profile can be specified in 1D (i.e. an elevation dimension specified for every unit length of the bridge) or 2D, whereby the profile is provided for multiple wheel lines (Liu et al., 2002). The road profile can be generated from field measurements of an actual roadway, or computer generated. For the latter case, the profile is often generated with the Power Spectral Density (PSD) function (Honda et al., n.d.). Standards have been developed for the function parameters based on road roughness categories, and while the profiles generated using these standards are useful, Loprencipe and Zoccali argue that they are no substitute for in-situ measurements when the behavior of a specific vehicle and structure is being investigated (2017).

The most common standard for describing and generating artificial profiles is the ISO 8608 standard. This standard describes the (spatial) frequency content of the profile with two terms that described a PSD function. The artificial profile is subsequently generated by summing a series of sine functions with amplitudes set according to PSD parameters (Tyan et al., 2009). These parameters include a waviness value (*w*) and the amplitude of the PSD function at a spatial wavelength equal to 10 meters (*C10*). The PSD amplitude of each spatial frequency band (*n*) is therefore assigned according to the following equation.

|  |  |
| --- | --- |
|  | (1) |

The amplitude of each frequency band in dimensional units is given by the following equation.

|  |  |
| --- | --- |
|  | (2) |

Where *Ω* is the angular spatial frequency in rad/meter, and *N* is the number of included frequency bins. The profile may then be generated with the following equation, where *ϕi* are the phase angles in radians.

|  |  |
| --- | --- |
|  | (3) |

Profiles should include approach roadway to allow for simulation and consideration of vehicle motion induced by the approach roadway profile. An approach length of 100 meters is common (González et al., 2011; McGetrick et al., 2013; O’Brien et al., 2014; OBrien and Keenahan, 2015).

### Solution Methods

There are different methods of solving the vehicle-bridge system, depending partially on the modeling methods employed. For simpler models, the equations of motion of both the vehicle and bridge can be explicitly derived. Their interaction can then be solved by coupling the equations or by using an iterative approach. For the coupled method, the equations of motion of the two systems are assembled into single mass, damping, and stiffness matrices for each time step (Kim et al., 2005, p.). Alternatively, for the iterative approach, the bridge and vehicle equations of motion are solved separately, and the force and displacement results of each used in the solution of the other (Wang and Huang, 1992, p.). The process is therefore repeated until convergence is achieved. Modal superposition can be used in the iterative approach and is also used in many commercially available FE software packages to drastically reduce the computational burden of solving the bridge response compared to time-step solutions (Au et al., 2001a; Yang and Lin, 2005).

## Influential Parameters for Dynamic Amplification

Many analytical and experimental studies throughout the past several decades have investigated which parameters are influential to dynamic amplification. These parameters have included: road surface condition, bridge span length and natural frequency, bridge type, vehicle speed, and vehicle weight and suspension characteristics. Of these, research has shown that road surface condition, vehicle speed, and vehicle weight and suspension type have the most effect on dynamic amplification.

### Bridge Parameters

Although some codes include span length or first natural frequency in their calculations of IM, studies have shown poor correlation between either of these parameters and dynamic amplification (Cantieni, 1983; Chang and Lee, 1994; Huang et al., 1993; Schwarz and Laman, 2001). Furthermore, although bridge type may have a significant impact of the dynamic behavior of a bridge, there are a wide variety of bridge types and even more varied structural characteristics within each type preventing any consistent relationship from being established (Deng et al., 2015).

### Vehicle Parameters

Vehicle speed has been shown to have a significant effect on dynamic amplification. Generally higher amplification factors result from higher speeds. In some studies, it has been shown that there is a critical speed for which the amplification factor is a maximum (González et al., 2010). However, although the various studies generally agree that vehicle speed is an important factor, the proposed relationships between vehicle speed and amplification factors is inconsistent, suggesting that this relationship is complicated and dependent on other factors (Deng et al., 2015). Similarly, vehicle mass and suspension characteristics have been shown to effect dynamic amplification, but again, no definitive relationship has been established. However, studies generally agree that the amplification factors decrease with increased vehicle weight and damping and with decreased suspension stiffness (Green et al., 1995; Nassif and Nowak, 1995).

### Road Surface Roughness

Previous studies have examined the impact that the road surface has on impact factors. Many analytical studies have shown that a rough road surface may result in higher dynamic amplification (Deng and Cai, 2010; Huang et al., 1995; Kim et al., 2007; Wang and Huang, 1992). However, the studies do not agree on the significance of the effect road surface has on dynamic amplification, which is likely due to variety of bridge types and geometry, road profiles, and model types employed.

Most analytical studies have used a single-line beam model in which the width of the bridge is reduced to a single beam with appropriate mass and stiffness. Some of the earliest simulation work was done by Aramraks at Purdue University. In his research, single span, 2-span and 3-span bridges were represented as single-line beam models. Surface roughness was idealized as a number of half sine waves, resulting in beam accelerations as much as 10 times those obtained with a smooth road surface (Aramraks, 1975).

As computing technology has progressed, more complex simulations have been carried out. A simply supported box girder was modeled with a moving mass over a rough road surface that was simulated by using PSD functions to produce a more realistic road profile, and obtained dynamic amplification factors (DAF) as high as 3.0 (Inbanathan and Wieland, 1987). Simulations of a 3-span continuous box-girder bridge (modeled as a beam) for a rough road surface generated a maximum DAF of 2.3 (Law and Zhu, 2005). Chatterjee et al. used a single line girder model of a continuous bridge, and showed that for certain combinations of speed and frequency ratios between the vehicle and structure, the DAF could exceed 4.0 (Chatterjee et al., 1994). The effect of long term deflections in addition to road surface roughness was investigated for a simple span and a 3-span prestressed concrete bridge and it was concluded that the long-term deflections had negligible effect on amplification factors, but the road surface roughness could cause amplification factors in excess of 2.0 (Au et al., 2001b).

The effect of road surface on bridge responses was further investigated with 3-dimensional FE models. Kou and DeWolf modeled a 4-span continuous plate-girder bridge, and examined the effect of smooth road surface versus 0.5 inch and 1 inch amplitude roughness and concluded that the road roughness had negligible effect on bridge deflections (Kou and DeWolf, 1997). In contrast, 3-D FE simulations of a 3-span non-continuous bridge by Li et al. showed that the surface roughness had a large effect on dynamic amplification, especially with increased speed. Based on simulation efforts, a maximum dynamic amplification of nearly 3.0 was reported for poor road condition and at 70 mph, while a DAF of only 1.2 was recorded from field measurements of the bridge (Li et al., 2008).

Indeed, the dynamic amplification factors obtained from field measurements are consistently lower than the factors suggested by analytical research and have similarly wide variation from bridge to bridge. Cooper instrumented two bridges in England and recorded a maximum DAF of 1.42. Cooper also created a probabilistic model of DAF based on field measured road roughness and span length that suggests a maximum mean DAF of 1.27 (Cooper, 1997). Park et al. examined the effect of road roughness on dynamic amplification by testing 25 highway bridges in South Korea. None of the bridges exhibited amplification factors greater than 1.25, but their results clearly showed that the amplification factors increased with the International Roughness Index (IRI) (Park et al., 2005). However, further research suggests that no single measure of road roughness can accurately predict DAF because of the many other influential parameters that contribute to DAF (i.e. bridge geometry, mass and stiffness; vehicle dynamic properties; vehicle speed; etc.). (Li et al., 2006; OBrien et al., 2006).

## Effects of Platooned Vehicles

Much research has focused on the load effect of congested traffic on bridges, whereby larger loads are applied to the structure due to the increased number of vehicles present. The increased load is exacerbated by the tendency of long lines of trucks to form (Han et al., 2015). While any traffic pattern of repeated trucks is often referred to as truck platoons, in this research platoons will refer to trucks that are virtually coupled by wireless communication and sensing that allows on-board computers to control headway and speed. Some research has looked into how this headway should be controlled to reduce congestion (Lipari et al., 2017), but little work has been done to understand the dynamic effect of platooned trucks on highway bridges.

In contrast, the problem has been extensively studied for railway bridges, where train cars present regular wheel spacing, vehicle spacing, and weight (Bolotin and Armstrong, 1965; Kurihara and Shimogo, 1978; Li and Su, 1999; Yang et al., 1997). Research has shown that railway bridges are susceptible to large dynamic amplifications (Wu et al., 2001), and that the amplification is dependent on such parameters as wheel spacing, train speed, bridge span length, and bridge first fundamental frequency (Kwark et al., 2004; Majka and Hartnett, 2008).

However, while the loading conditions of railway bridges are similar to that induced by platooned trucks, the structural system is markedly different. The tracks of railway bridges are often isolated from the structure by ballast, providing different load distribution and significantly higher damping than is accomplished by the deck of a highway bridge. Furthermore, the track of a railway bridge is typically smoother and with fewer irregularities than a highway bridge roadway.

## Bridge Vibration Limits

The dynamic effect of traffic on bridges is not just of concern for amplification factors or strength limit states (i.e. rating factor). Excessive vibrations may result in reduced fatigue life, as the vibrations can result in more stress cycles per loading event. Furthermore, structural vibrations should be limited so as not to upset the bridge users.

Past studies have looked at what constitutes objectionable vibrations and the characteristics that influence a human’s perception of those vibrations. In general, these studies concluded that as the displacement and frequency increase, the vibration is considered more intolerable to users. However, there is no single parameter that can predict a human’s perception of a given vibration (Gaunt and Sutton, 1981). Some of the earliest work on this topic was carried out by Reiher and Meister. They developed sensitivity curves by subjecting people to vertical harmonic vibration (Reiher and Meister, 1931). These sensitivity curves still provide an acceptable characterization of human perception to vibrations.

AASHTO bridge specifications state that bridges should be designed to avoid psychological effects and that acceleration is the primary factor for human sensitivity to bridge deformations, but fails to provide any specific limits for vibrations (AASHTO, 1998). Instead they placed limits on span-to-depth ratios and live load deflections in hopes that this would prevent unsatisfactory dynamic behavior. The Ontario Highway Bridge Design Code of 1983 introduced a new serviceability limit state that was meant to control vibrations that would be objectionable to pedestrians by restricting deflections based on the first frequency of the structure (Csagoly and Dorton, 1978). The Eurocode also advises bridge designers to limit vibrations to avoid the discomfort to users, but again, fails to provide guidance as to how that may be accomplished (BS, 2006).

## Knowledge Gaps

Much study has been devoted to understanding the factors that affect dynamic amplification and to the development of methods for calculating it. However, numerous bridges have been identified as having amplifications far outside those predicted by established methods, thus suggesting there are mechanisms whose effect on dynamic behavior is still poorly understood. Even though past studies have identified influential parameters that may contribute to the apparent outliers, there has been no consensus on how to incorporate those parameters into dynamic response predictions. This is due, in part, to the failure of studies to consider all influential parameters simultaneously and thus their interdependency. Furthermore, much of the previous research considered only simple loading models (i.e. design trucks) that cannot possibly account for the variety of vehicles that any given bridge experiences every day. With the advancement of computational tools and processing power, a comprehensive parametric study is only recently feasible.

Dynamic amplification factors, due to the nature of their formulation, are incapable of applying to all structures. They were developed by applying statistical methods to a population of bridges that were selected for investigation (Fenves et al., 1962), and therefore, while applicable to a majority of that population, cannot be expected to remain accurate for every single bridge in the population. Furthermore, there is no assurance that the chosen sample sets are representative of the entire bridge stock, nor can it be assumed that the characteristics of the entire bridge stock are time invariant.

There is therefore a need, in the interest of developing methods whose range of applicability includes a larger portion of the bridge stock, to investigate the effect and interdependency of a more comprehensive set of parameters and to identify and characterize those bridges that are “outliers” and non-conservative. Furthermore, changes in bridge design, loading, maintenance practices, etc. and their influence on population characteristics should be considered in an effort to ensure resulting conclusions will continue to be applicable.

As truck platoons are introduced to America’s highways, the lack of understanding of vehicle-bridge interaction could further threaten the bridge stock. To date, research on the impact of repeated vehicles has been limited to railway bridges, for which the loading and structural characteristics are significantly different. Before this novel mode of transportation is implemented, investigation of its impact on bridge response should be conducted for which the bridge and load patterning are represented in a manner that is realistic of actual structures and truck platoon scenarios.