Chapter 4 – Dynamic Load Allowance

Authors: Eugene OBrien; Jennifer Keenahan; Ales Znidaric; Jan Kalin

4.1. Introduction

4.1.1 The Phenomenon

Bridges vibrate in response to vehicle crossing events and this can result in an increase in traffic load effects (LEs). The phenomenon is illustrated in the very simple example of Figure 4.1. This shows the components of strain at the bottom of the beam in the center of a simply supported span, caused by the passage of a single point force. The static component is generally the largest, and in this case is triangular, peaking when the force passes mid-span (Figure 4.1(b)). The dynamic component, Figure 4.1(c), is approximated as a sinusoidally varying vibration at the bridge's first natural frequency. This example shows that dynamics can be adverse when this component has reached a peak in the time it takes for the load to reach the center of the bridge – Figures 4.1(c) and (d). Equally, the dynamic component may have reached a trough in which case, dynamic 'amplification' can reduce the total LE – Figure 4.1(e). Thus, the vehicle can be taken to have a 'pseudo-frequency' associated with its speed that can resonate or not with the bridge's first (or other relevant) natural frequency. If dynamic amplification is plotted against vehicle speed, a series of peaks can be seen, corresponding to this resonance effect.

Figure 4.1 Mid-span strain due to a point force crossing a beam: (a) Point force on bridge; (b) Static response; (c) Dynamic response for positive DAF; (d) Total (static + dynamic) response for positive DAF; (e) Total response for negative DAF.

When two point forces, corresponding to a 2-axle truck, cross a beam, the static peak at mid-span strain occurs when one of the forces passes mid-span. If the bridge is again assumed to vibrate at its first natural frequency, starting when the first axle arrives on the bridge, a similar situation occurs but there are two possible resonance events – a peak in bridge vibration could coincide with the arrival of either axle at mid-span. This theory was first proposed by Frýba (1996), a pioneer in the field, and the resonance effect was illustrated using simple models by Brady et al (2006) and Brady & OBrien (2006). However, when the results of this simple theory are compared with field measurements, there is little agreement – real vehicle/bridge interactions are far more complex.

In practice, a vehicle is not well represented by a series of point forces. A vehicle has mass and the movement of the mass will change the system's natural frequencies during the crossing event, especially for shorter bridges. The mass of a vehicle is supported on a number of springs and there are associated natural frequencies. Heavy vehicles generally have body bounce and pitch frequencies around 1.5 to 4 Hz and axle hop and suspension pitch frequencies around 8 to 15 Hz (Cebon 2000). The vehicle and the bridge interact dynamically during the crossing event and this interaction can be constructive or destructive. For example, if the bridge is moving downwards when an axle arrives, this movement tends to be amplified by the axle (constructive interference). If on the other hand, the bridge is moving upwards when an axle arrives, the vehicle and the bridge act against one another (destructive interference). All of this is complicated by the presence of the road (or rail) surface profile. The natural variations in a typical road surface are of the same order of magnitude as bridge deflections and cover the entire spectrum of frequencies. Hence, as a vehicle crosses a bridge, a wide range of motions are going on and several frequencies are at play.

OBrien et al (2010) and Caprani et al (2012a) propose a statistical approach to dynamics. They point out that it is not the ratio of total (static plus dynamic) LE to static LE for a single loading event that is relevant. Given that statistical studies of static loading are generally carried out first and an allowance for dynamics is applied afterwards, they suggest that a more relevant parameter is 'Assessment Dynamic Ratio' (ADR): the ratio of characteristic maximum total LE to characteristic maximum static LE. They calculate characteristic total LE by performing vehicle/bridge dynamic interaction analyses on large numbers of extreme loading events. The trend is quite interesting. While dynamic amplification for some non-critical loading events can be quite large, the trend amongst the extreme loading events is much more modest, with the calculated ADR typically not exceeding 10%. Znidaric (OBrien et al 2013a) directly measures static and total LEs and provides further evidence that the influence of dynamics tends to diminish as static loading events become more extreme. This can be explained in part by the increasing complexity of extreme loading events, particularly those involving multiple vehicles, which increases the likelihood of destructive interference effects and tends to reduce the total LE.

Bridge loading standards generally make some allowance for dynamic amplification. This can depend on the type of load effect (LE), the span length, the number of lanes, etc. The notional load model for which bridges are designed, generally includes an allowance for dynamic amplification.

4.1.2 Basic Definitions

As outlined above, the load effects that a vehicle generates in a bridge arise from a combination of the static weight of the vehicle and inertial loads due to dynamic interaction between the vehicle and the bridge. The consequent total load effect, ε_T, is typically larger than that from a wholly static analysis, ε_S. The dynamic amplification factor (DAF) represents the ratio of these two load effects and is defined as:

$$\mathrm{DAF} = \frac{\varepsilon_T}{\varepsilon_S} \tag{4.1}$$

An alternative definition of the same concept, used by some authors (e.g., McLean and Marsh 1998) and codes of practice (AASHTO 2012, Standards Australia 2017, 2017a), is the Dynamic Load Allowance (DLA), Dynamic Increment (DI), or Impact Factor (IF), given in % by:

$$DLA = DI = IF = \left(\frac{\varepsilon_T - \varepsilon_S}{\varepsilon_S}\right) \times 100 \tag{4.2}$$

4.1.3 Factors influencing dynamic amplifications

As will be seen later, codes of practice take very different approaches to the consideration of dynamic effects. This is because of the complexity of the phenomenon and the vast array of factors that contribute to it. These are the main contributing factors, and each is explained in more detail below:

- The road surface;
- The bridge structure;
- The vehicle(s);
- The loading event;
- The structural response (LE) under consideration.

The road surface condition is consistently found to be a strong indicator for the level of dynamic interaction that occurs (Deng et al 2014). Smooth road profiles produce lower DAFs than rough road profiles. Many authors have explored the relationship between road surface quality indices, such as the International Roughness Index (IRI), and the resulting DAFs (e.g. dynamic amplification estimator, bridge roughness index, etc.). However, due to the many other factors affecting DAF, no general relationship has been found. However, one certainty that can be stated is that regular pavement maintenance to ensure a smooth profile reduces the level of DAF, with resulting benefits for bridge maintenance needs. This is a cost-effective approach to increasing bridge safety. In doing so, particular attention should be paid to bumps near the start of the bridge.

The bridge structure itself has an important role in the level of DAF that occurs. There is a strong inverse relationship between bridge length and first natural frequency (e.g. Chan and O'Connor 1990). Hence one is often used as a proxy for the other, as will be seen across various codes of practice. The literature (see Deng et al 2014, Li 2006) is consistent in suggesting that short bridges are subject to higher dynamic interactions than longer bridges. One exception is when frequency matching occurs between the vehicle and bridge, a higher DAF occurs. Most research has been done on short to medium span bridges (around 4 – 8 Hz) due to their prevalence and higher dynamic interactions due to frequency matching. The very common I-girder bridge form seems to be the most studied (Deng et al 2014).

The main indicator of the level of dynamic interaction is the level of static load carried by the girder: girders that are more highly loaded statically, tend to have lower DAFs. However, as with all vibration problems, increased damping usually leads to reduced response, and DAF is no different. Timber bridges (for example) are found to have lower DAF than steel bridges (for example). Interestingly new materials, such as fibre-reinforced polymer (FRP) do not exhibit the expected trend, and further research is needed on these forms of construction (Deng et al 2014). Finally, Rattigan (2007) studies the influence of pre-existing vibration of the bridge (e.g. from a preceding loading event) on the resulting DAF of a new loading event. In some cases, the dynamic increment can more than double, but this requires a very particular set of circumstances (inter-vehicle gap and damping).

The vehicle or vehicles comprising the loading event are significant influencers on the level of dynamic interaction. By far the most significant influence is the mass of the vehicle. In particular, the ratio of the vehicle to bridge mass determines the degree of interaction between the two of them. A high ratio changes the overall dynamic properties of the system, and also results in higher static stress. It is widely found that dynamic amplifications decrease with increasing weight of vehicles or static stress (see, e.g. Figure 4.14). Many authors have investigated the influence of vehicle weight or static stress on DAF: Hwang and Nowak (1991), Huang et al (1993), Chang & Lee (1994), Abdel-Rohman & Al-Duaij (1996), Kim & Nowak (1997), DIVINE (1997), McLean & Marsh (1998), Laman et al (1999), Broquet et al (2004), SAMARIS (2006), Li et al (2008). Different vehicle types also exhibit different responses. For example, Cantero et al (2011) examine the DAFs for a range of bridges subject to heavy articulated 5-axle trucks or cranes, allowing for bumps at the start of the bridge and for meeting events. For shorter spans (< about 20 m) the articulated truck yields higher average DAFs than the cranes, but this difference is less evident for longer spans.

One of the most frequently studied issues is the effect of vehicle velocity on DAF as there is the potential for frequency matching between the crossing pseudo-frequency (velocity/length) and bridge frequency. Indeed, it has been shown that very high levels of DAF can occur, but usually only at very high speeds. However, these studies are usually theoretical and simplified, and when more realistic simulations (or field tests) are conducted, these predicted high DAFs are difficult to recreate (Zhu & Law 2002). Vehicle accelerations and decelerations are also found to affect DAF, with increased DAFs found in some cases for high decelerations (Deng et al 2014, Li 2006). Different truck suspension systems are known to influence the dynamic amplification. For example, the DIVINE (1997) project reports that air suspension causes less (5-10%) dynamic amplification of static wheel loads than spring-leaf suspensions (20-40%). Indeed, Harris et al (2007) show how a bridge-friendly vehicle suspension system can be designed that minimizes dynamic amplification.

Despite the body of research, there is no clearly identified link between DAF and the number of axles (Deng et al 2014). Presumably, this is because it is difficult to isolate the effect of axle number from other properties such as vehicle weight. In both field trials and numerical simulations, it is important to have sufficient approach length of the vehicle before the bridge so that initial excitation can take place. This is merely to ensure that the calculated or measured DAFs are realistic. Finally, the transverse location of the vehicle in its lane affects DAF but only insofar as it affects the level of static load in any particular girder or element due to the transverse stiffness of the deck. Then, the reduction of DAF with increasing static load is again observed, and vice versa (Huang et al 1993, Deng et al 2014).

The loading event is characterized by the number of vehicles and the inter-vehicle spacing. The literature consistently shows that the DAF for multiple truck presence tends to be lower than for single trucks (Deng et al 2014, AustRoads 2003). This is related to the increased static load (and consequent lower DAF) that has been noted previously and to the likelihood of destructive interference. For example, González et al (2011) find that for two 5-axle trucks meeting on any bridge over 12 m, the largest DAF for shear is 5% and for bending is 1%. Similarly, Rattigan uses a calibrated finite element model of the 32 m long Mura River Bridge (SAMARIS 2006) and finds that in the worst case, two 5-axle truck meeting events, give a DAF of 15%. In terms of the inter-load spacing, theoretical studies have found that for some idealized situations, a higher DAF can result for multiple moving point loads at particular spacings, than for a single point load (Li 2007). However, for more realistic scenarios, DAF is found to consistently reduce due to the presence of more 'load' on the structure.

The final major factor governing the DAF is the structural response or LE considered. There are many LEs that may be of interest, such as bending moment, shear force, torsion, deflection or stress. The reported DAFs vary, depending on the response selected, even for the same bridge and vehicle (e.g. Huang 2008, González et al 2011). Validation of simulation models against field trials is commonly conducted, but these usually only report on strain or deflection DAFs. Therefore, simulation models (validated as far as possible) are often used for other effects. Interestingly, the critical location for static LE is not necessarily that for critical total (static + dynamic) LE (Li 2006). Cantero et al (2009) find that the maximum total bending moment occurs away from the mid-span of a simply supported beam, and define a 'full' DAF (FDAF) as the ratio of the maximum total response anywhere on the beam to the mid-span static bending moment. They find this to be greater than DAF and greater than unity (unlike DAF). Huang (2008) finds similar results for a curved box girder. In short, it matters where, on the bridge, DAF is measured.

4.2 Codes

Codes of practice and standards internationally take different approaches to the issue of dynamic vehicle/bridge interaction. In general, codes seek to simply the approach and limit the number of contributing factors considered. A selection of approaches is summarized in Table 4.1.

Table 4.1 Summary of some current international codified rules for dynamic allowance (*DLA* = Dynamic Load Allowance, *L* = loaded length in m, *f* = frequency) (Deng et al 2014)

Country/Region	Contributing factors	Summary of Allowance
United States[1]	Bridge length	*DLA* = 0.33 of truck portion of load
Europe[2]	Bridge length, no. lanes	$DLA = 0.3 - 0.2\left(L/50\right)$ $L \le 50$ m $DLA = 0.1$ $L > 50$ m
Australia[3]	Number of axles	$DLA = 0.1 - 0.4$
China (MTPRC)[4]	Frequency	$DLA = \begin{cases} 0.05 & f < 1.5 \text{ Hz} \\ 0.1767 \ln f - 0.0157 & 1.5 \le f \le 14 \text{ Hz} \\ 0.45 & f > 14 \text{ Hz} \end{cases}$
Japan[5]	Length + Material	$DLA = \dfrac{20}{50 + L}$

Canada (CHBDC)[6]	Number of axles	$DLA = 0.25 - 0.5$

[1] AASHTO 2012, [2] CEN 2003, [3] Standards Australia (2017), [4] MTPRC 1989, [5] JRA 1996, [6] CSA 2006.

4.2.1. AASHTO

The dynamic load allowance in the AASHTO specifications up to 2007, called the Impact Factor, was specified as a fraction of the static live load and as a function of the loaded length:

$$DLA = 15.24/ (L + 38.1) < 30\%$$

where L = loaded length in metres. In the AASHTO LRFD Specifications, used since 2007, dynamic load allowance is also specified as a portion of the static load, but independent of loaded length. Since the 2012 edition, for the Strength Limit States, the DLA is specified as 33% of the effect of the design truck only, with no dynamic allowance added to the uniformly distributed portion of live load.

The design dynamic allowances are based on field measurements. These indicated that the dynamic component is largely independent of vehicle weight. As static load is directly related to vehicle weight, the DLA reduces as ever heavier vehicles are considered. DLA is additionally reduced for simultaneous occurrence of two or more trucks in adjacent lanes. The maximum observed DLA in the measurements on about 100 bridges in Michigan was less than 20% for a single truck and less than 10% for two trucks side-by-side.

4.2.2. Eurocode

The Eurocode for bridge traffic loading, EN 1991-2:2003, has been adopted throughout the European Union and several other countries, worldwide. Factors, sometimes called ‘α-factors’, are allowed to vary between countries, regions and road networks within countries. The α-factors and other issues specific to a particular country, are specified in a national annex.

The allowance for vehicle/bridge dynamic interaction is not explicit in the Eurocode but an allowance was incorporated during the development of the standard. Thus the notional load model for normal traffic, Load Model 1, is deemed to represent a characteristic maximum 1000-year value, including an allowance for dynamics, as appropriate. These built-in allowances are presented in Figure 4.2. It can be seen that they depend on the bridge length, the load effect (bending moment or shear) and the number of lanes of traffic. In multi-lane traffic, there is an increased likelihood of destructive interference, which reduces the extreme response. Clearly, in the development of the standard, these dynamic allowances were only applied to extremes of free-flowing traffic. For extremes of jammed traffic, it is generally assumed that no allowance is required.

Figure 4.2 – Eurocode allowances for dynamics

4.2.3. Australian Standard

Australian Standard, AS 5100, specifies dynamic load allowances for vehicles traversing Australian road bridges. Part 2 of the code (AS 5100.2:2017) is used for the design of new bridges, whereas Part 7 (AS 5100.7:2017) is for the assessment of existing bridges. For the design of new bridges, the code specifies a range of loading arrangements, referred to as SM1600 loads, as shown in Table 4.2. The most critical load will depend on the bridge loaded length. It specifies a single deterministic DLA for each of these design loads that ranges from 0.0 to 0.4, depending on the number of axles acting on the bridge. Unlike most other international codes, load effect type, span length, and bridge frequency are not considered.

Table 4.2 Dynamic load allowances for Australian design load model, SM1600

Load Model Component	*Number of Axles*	*DLA*
W80 Wheel Load	1	0.4
A160 Axle Load	1	0.4
M1600 Moving Load	3	0.35
M1600 Moving Load	12	0.3
S1600 Stationary Load	12	0.0
Heavy load platform	16	0.1

For bridge assessment, the DLA is specified as 0.4, with some exceptions. If, in the assessment, one of the SM1600 loads is selected as the traversing vehicle, then the value pertaining to that load (Table 4.2) prevails. Alternatively, if the road roughness is suitably low, a DLA of 0.3 may be adopted. Finally, the DLA can be reduced to 0.1 or even ignored (0.0), based on a reduced speed for the assessment vehicle.

The dynamic load allowances of AS 5100 were developed from a parametric study, considering single vehicle events on a small subset of representative Australian bridges (Austroads 2003). While it is acknowledged that multiple vehicle events are the critical load cases for many bridges, it is deemed appropriate to use a DLA based on a single vehicle, since the SM1600 loads are single vehicles. The study concludes that a relationship between dynamic load allowance and bridge frequency is present, but sees it as secondary to the influence of road roughness.

4.2.4. Chinese Standard

In the Chinese code for highway bridges (JTG D60-2015), the dynamic effect induced by vehicles on structural components is taken into account using a so-called impact coefficient, μ, whose definition is the same as the DLA.

In principle, the value of the impact coefficient is determined from the natural frequency of the structure, but other factors are also taken into account when determining the value, including the type of structural component and whether the LE is local or global. The design of superstructure elements in steel, reinforced concrete and masonry arch bridges, together with supporting bearings and reinforced concrete abutments, should include the dynamic effect. Other bridge types are deemed to be unaffected by dynamics, including arches with pavement/fill thickness of over 0.5 m, culverts and gravity abutments. The impact coefficient, as found from the impact coefficient formula, is shown in Figure 4.3.

Figure 4.3 Impact coefficient (DLA) for China, as a function of fundamental frequency

4.3. Statistical Approach to Dynamics

Most codes apply dynamic amplification factors to characteristic maximum static LEs or a notional load model deemed to represent these. If the worst static and dynamic cases are combined, such an approach is conservative because it does not recognize the reduced probability of two extremes (static and dynamic aspects) occurring simultaneously. The best example of this is that there are often very large dynamic amplifications for light trucks but much lesser ones for heavy trucks. Bridge-truck(s) interaction is sufficiently complex that the dynamic component of the LE may be considered as a random variable. Therefore, with any given crossing event, there are two resulting processes: static and total LE. For bridges, it is critical combinations of these two processes that are of interest.

Multivariate extreme value theory can be used to analyze critical combinations of several processes (Caprani 2005, González et al 2008). Such an approach is more reasonable as it includes the respective probabilities of occurrence as well as any relationship between them. This theory is used here to incorporate the dynamic interaction of the bridge and trucks into an extreme value analysis for total LE. The results of this analysis can be used to determine Assessment Dynamic Ratio (ADR), a dynamic allowance that may be applied to the results of static simulations to determine an appropriate maximum total LE.

4.3.1 Assessment dynamic ratio

Assessment Dynamic Ratio (ADR) is defined as the ratio of characteristic maximum total load effect, $\hat{\varepsilon}_T$, to the characteristic maximum static value, $\hat{\varepsilon}_S$:

$$DAF = \frac{\hat{\varepsilon}_T}{\hat{\varepsilon}_S} \tag{4.3}$$

where $\hat{\varepsilon}_T$ includes both static and dynamic components of the LE. The key point here is that the characteristic maximum static and the characteristic maximum total do not, in general, correspond to the same loading events. In fact, they represent levels of the load effect rather than to particular loading events.

The concept is illustrated in Figure 4.4 where each point is a maximum-per-100-year loading event. The maximum static and maximum total LE in the dataset are the same – point A – and DAF is the slope of a line from the origin to this point. Similarly point B is the 2nd greatest static and 2nd greatest total LE. However, the 3rd greatest static LE is point C whereas the 3rd greatest total is point D. The corresponding ADR value is the slope of a line from the origin to point E and does not correspond to any particular loading event. ADR is, in effect, the ratio of the characteristic maximum total LE needed for design or assessment to the characteristic maximum static value, which can be found using the methods described in Chapter 3. In general, ADR is considerably less than the largest DAF values, which tend to be associated with less critical LEs such as point F.

Figure 4.4 – Maximum-per-100-year load effects (dN/mm^2) (Caprani et al 2012a)

To illustrate the ADR concept example, Caprani et al (2012a) use the Slovenian Mura River Bridge and WIM data from the A6 motorway near Auxerre in France. A finite element model of the bridge is developed in which dynamic behavior of the model is calibrated against measured responses for single and two-truck meeting events.

For this site, 10 years of bi-directional, free-flowing traffic data is generated numerically, and this traffic is passed over the influence line for one of the girders to determine the static LEs that result. Each year of simulation is broken down into 'months' of 25 working days each and there are thus 10 such months in each year of simulation (assuming 250 working days per year). As a basis for further analysis, the events corresponding to monthly-maximum static LE are retained. This is done to minimize the number of events that are to be dynamically analyzed, as well as providing a shorter extrapolation 'distance'.

The 100 monthly-maximum loading events obtained from the static simulations are analyzed using finite element bridge-truck interaction models. Dynamic simulations are carried out and the results of the simulations is a population of 100 monthly-maximum loading events for which both total and static LEs are known. Scatter plots such as Figure 4.4 are drawn to illustrate the results.

To capture the relationship between the static and total LE values, a bivariate extreme value distribution is fitted to the data, in this case a Gumbel logistic bivariate distribution. The results are shown on the contour plot of probability density in Figure 4.5. It can be seen that the most probable results are around (ε_S, ε_T) = (68, 71). The skew in the contours reflects the correlation between static and total – as static LE increases, total LE also tends to increase. As probabilities get less (outer contours), the correlation continues: for example, the last contour corresponds to maxima of (ε_S, ε_T) = (76, 81). For most loading events, total exceeds static, i.e., DAF > 1.0, but this is not always the case, especially for less extreme loading events.

Figure 4.5 – Probability density contours for a bivariate distribution fitted to maximum-per-month data (Caprani et al 2012a).

4.3.2 The shift in probability paper plots due to dynamics

OBrien et al (2010) model the dynamic interaction between vehicles and a bridge using a 3-dimensional vehicle model traversing a finite element plate model, which takes into account the road surface roughness and characteristics of the truck fleet such as speed, weights and suspension properties. Traffic simulations were used to generate 100,000 different annual maximum loading scenarios, 10,000 for each of 5 bridge lengths and 2 lane factors. Each of the 100,000 annual maximum events were analyzed using a vehicle-bridge interaction model for two ISO road classes ('A' and 'B') and two expansion joint conditions (healthy and damaged), adding up to a total of 400,000 dynamic analyses. The road profile and vehicle parameters were varied randomly within a Monte Carlo simulation scheme. The bi-directional traffic traversed the bridge and the critical loading scenarios were found to be made up of anything from a single vehicle event to a combination of three vehicles.

The ADR values for 5, 50, 75 and 1000 year return periods are inferred by fitting the generalized extreme value distribution (GEV) to the top 30% tail of the annual maximum data. Figure 4.6 plots static and total bending moment for the 45 m bridge on Gumbel probability paper, together with the GEV tail fits. The studied return period levels of probability (5, 50, 75 and 1000 years) are shown. As can be seen, the distribution fits show excellent agreement with the data. The shift in the probability paper plot due to dynamics is clear, with the two curves being close to parallel. The ADR is the ratio of total LE to static, for a given level of probability. For example, for the 75-year return period, ADR is the ratio of bending moment corresponding to point B to that corresponding to point A.

Figure 4.6 – GEV fit of data, static (+) and total (×) bending moment on Gumbel probability paper, for 45 m span, high lane factor, Class 'B' profile (OBrien et al 2010)

The simulation results for mid-span bending moment in a range of spans are illustrated in Figure 4.7. The ADR values are quite small, with almost all results giving a value of less than 1.05. The road roughness has an effect, with Class B (ASTM 2015) road surfaces giving higher factors than Class A, typical of a well-maintained highway. The return period chosen has no significant effect. This is because the statistical trends of static and total LE are similar on probability paper (Figure 4.6), with the curves being roughly parallel. OBrien et al (2010) use Finite Element analysis to derive two extremes in the lane factor to allow for the degree of load sharing between lanes. A 'high' lane factor of 1.0 is used to represent cases where the secondary lane is close to the beam of interest and/or the bridge is transversely stiff. In this case, vehicles in both lanes contribute equally to the bending moment. A 'low' lane factor of 0.45 represents the case where the secondary lane is remote from the beam of interest and/or the bridge is transversely flexible. In this case, a vehicle on the secondary lane is deemed to contribute only 45% of its bending moment to the beam of interest. A comparison between Figures 4.7(a) and (b) does not show any significant difference in the ADR trends as a result of the lane factor assumed.

(a)

(b)

Figure 4.7 ADR values for 75 (- - - - - -) and 1000 (··················) year return periods; for class 'A' (•) and class 'B' (○) profiles; (a) High lane factor; (b) Low lane factor

4.3.3 The contribution of surface roughness to dynamics

The condition of the road surface is another factor influencing the response of a bridge to a passing vehicle (DIVINE 1998) as well as specific locations of potholes and expansion joints (SAMARIS 2006). It is common to have a significant discontinuity at expansion joints as they are frequently the weak points of bridges and easily damaged (Lima & Brito 2009) or because differential settlements occur between the bridge and the abutment. A number of authors (Michaltsos 2000, Michaltsos et al 2000, Li et al 2006) show how a single pothole or bump, positioned at a critical location, can generate a large dynamic amplification. In particular, Chompooming & Yener (1995) show that combinations of pothole characteristics (i.e., height and length) and vehicle speed can result in very high dynamic effects.

Clearly a bump in the right location can cause an axle or axle group to be in a 'down bounce' just when the static load effect is maximum.

OBrien et al (2010) calculate Assessment Dynamic Ratio (ADR) for a wide range of spans, LE's, deck transverse stiffnesses, etc. Following a review of expansion joint surveys in Japan (Kim et al 2007) and Portugal (Lima & Brito 2009), they considered the presence of a 20 mm deep depression over a 300 mm length, 500 mm before the center line of the bearing. Figure 4.8 shows the resulting 'Bump Dynamic Increment', defined as the difference between the ADR calculated in the presence of the bump and the ADR calculated in its absence. Positive values indicate that ADR increases in the presence of the damaged expansion joint. It can be seen in the figure that the influence is only significant for the shorter spans and even then it does not exceed about 1.5%. For larger bridge spans, the bump makes little or no contribution. This is because the influence of the bump dissipates quickly and, for longer spans, its effect is negligible by the time the truck reaches a critical point on the bridge. Clearly bumps nearer to that critical truck location would be more significant.

Figure 4.8 – Bump Dynamic Increment for high lane factor and 50 year return period for class 'A' (•) and class 'B' (○) profiles (after OBrien et al 2010)

4.4 Field measurements

Field measurement of dynamic amplification has clear advantages as it takes account of all the uncertainties simultaneously – vehicle, bridge and road surface properties. A limitation is that it cannot provide the dynamic amplification appropriate for design directly, such as the ADR. In a number of cases, dynamic amplification of bridge response was measured for a finite number of known heavy vehicle crossings (Cantieni 1983, Deng et al 2015). Measurement provides a good indication about bridge dynamic behavior under specific traffic loading and was used extensively in the past to account for the dynamic part of loading in bridge design codes. However, there are some drawbacks. The first of these is that each vehicle has specific dynamic characteristics depending at least on its type, gross weight, axle load distribution, axle configuration and speed. This variability is a problem if the experiments only provide responses to a limited number of possible vehicles. Secondly, the vehicles used for such tests were typically not loaded above the legal limits. Thus, the dynamics of extreme illegally overloaded vehicles were not captured. Thirdly, responses to multiple truck presence events, and extreme overloaded vehicles are practically impossible to measure. This means that the field measurements are necessarily biased towards common vehicle types and tend to give conservative values for DLAs. To collect reliable and useful information about the dynamic behavior of bridges under random traffic, it is desirable that the bridge dynamic response be measured for many thousands of random vehicles and multiple vehicle presence events. This is only practical if bridges are instrumented with sensors that measure the total response and the information is coupled with vehicle loading information, typically provided by a weigh-in-motion system.

4.4.1 History of using Bridge Weigh-in-Motion to estimate dynamic amplification

A bridge weigh-in-motion (B-WIM) system uses an instrumented bridge as a weighing scale to estimate the static weights of passing vehicles. B-WIM systems can be used to find static and total responses simultaneously and hence to estimate the dynamic amplification. The first known attempt to measure the dynamic behavior of bridges with a B-WIM system was done by Ghosn & Xu (1989). They successfully show the potential of an extended B-WIM algorithm, on some bridge types, to estimate dynamic response under random traffic. A few years later Nassif & Nowak (1995, 1996) took more extensive measurements of dynamic behavior under random traffic. They coupled a dedicated acceleration measurement system with a commercially available B-WIM system to test four bridges with spans from 9 to 24 m and evaluate the dynamic amplification by comparing the dynamic signal with the static B-WIM approximation. The latter was obtained by filtering the dynamic signal with a Fast Fourier Transform. The filtering parameters were selected based on the experience of the researchers. The dynamic effect was described with a Dynamic Load Factor (Equation 4.1).

Nassif & Nowak (1995, 1996) approximated the static strain with a filtered total strain response. A few tens of DLFs were calculated on each bridge, for each individual girder, and were presented as a function of maximum static stress. Nassif & Nowak conclude that the dynamic component of stress or strain (i.e. the dynamic increment, ε_D) is practically independent of the static component and that, as a result, the DLF decreases with increasing static stress/strain. Furthermore, they note that for very heavy trucks, the DLF does not exceed the theoretical results. Two years later, Kirkegaard et al (1997) came to similar conclusions by calculating the dynamic impact factors from selected simulated truck crossing scenarios.

More extensive attempts to use B-WIM for the calculation of dynamic amplifications have been made possible with the further development of B-WIM algorithms, which can provide the dynamic amplification for loading events in real time (Žnidarič et al 2008). In this study, the static response is estimated (dashed curve in Figure 4.9) as the sum of the contributions of each axle, calculated using the bridge influence line and the axle weights estimated by the B-WIM system (dotted curves in Figure 4.9). The disadvantage of this method was that any inaccuracy in the axle load estimates could have a significant influence on the accuracy of the static response and hence on the inferred DAF. The two main reasons for errors in B-WIM axle load calculations are the ill-conditioning of the system of equations (OBrien et al 2009a) and misidentification of axles (Žnidarič et al 2017). Unfortunately, the likelihood of miscalculation of axle loads increases in cases where the DAF is likely to be significant, i.e. on bridges susceptible to dynamic excitation from traffic loading. Therefore, DAF values measured in this way, especially the higher values, have to be treated with caution.

Figure 4.9 Measured response of a bridge to a 4-axle truck in Volts (proportional to strain), including total (solid curve), estimated static (dashed), response to individual axles (dotted) and relative axle locations in time (solid spikes)

In the *ARCHES* project (González et al 2009), thousands of DAF values on several bridges were calculated and measured, through simulation and B-WIM measurement. A similar reduction in dynamic allowance with vehicle weight and corresponding load effects was observed as in Nassif & Nowak's (1995, 1996) study.

4.4.2 DAF results inferred from Bridge Weigh-in-Motion

In recent years, an alternative method has been developed to calculate DAF values with a B-WIM system. Similar to Nassif & Nowak (1995, 1996), it calculates the static approximation of bridge response by filtering the measured dynamic signal with a Fast Fourier Transform (Kalin et al 2020). The significant advancement compared to previous attempts is that it defines the optimal filtering parameters from a few tens or hundreds of bridge responses to random passing traffic, and then calculates the DAF values automatically for all bridge crossing events. This approach removes the need for an expert who selected the filtering parameters based on personal experience.

A synthesis of the recent DAF/DLA results calculated from B-WIM measurements is shown in Table 4.3 (Kalin et al 2020). It compares results from 12 Slovenian and 5 US bridges, with influence line lengths between 5.5 and 35.0 m. On sites SI04 and SI06, the two opposite lanes were treated separately and marked as ‘a’ and ‘b’ in the table. The bridge superstructure types were of two major groups: B&S - beams/girders with reinforced concrete (RC) deck and RC slabs. Measurements took place from a few days to almost two years and resulted in a few hundreds to almost 750 000 DAF results.

Table 4.3 Summary of the sites and results of the analysis

Site	**Type**	DAF

		Length [m]	No. Vehicles	Mean	σ	Max	at GVW [t]
SI01	Slab	7.4	202	1.23	0.14	1.96	13.7
SI02	Slab	10.5	4,590	1.10	0.10	2.31	39.8
SI03	Slab	12.0	1,850	1.11	0.08	1.80	38.8
SI04a	Slab	5.5	516	1.03	0.03	1.23	24.6
SI04b	Slab	5.5	318	1.04	0.03	1.14	8.2
SI05	Slab	6.8	617	1.08	0.05	1.40	5.1
SI06a	Slab	9.7	865	1.10	0.06	1.35	8.6
SI06b	Slab	9.7	474	1.19	0.12	1.48	15.7
SI07	Slab	6.6	746,594	1.09	0.04	1.39	17.9
SI08	B&S	34.4	432,307	1.06	0.05	2.39	22.5
SI09	B&S	35.0	402,595	1.02	0.03	1.56	5.3
SI10	B&S	25.8	289,533	1.05	0.03	1.55	10.0
US01	B&S	12.2	1,979	1.05	0.04	1.34	8.3
US02	B&S	10.4	15,295	1.08	0.08	1.86	13.4
US03	B&S	11.0	7,398	1.05	0.06	1.75	18.4
US04	B&S	10.4	1,608	1.08	0.08	1.81	20.9
US05	B&S	25.0	25,219	1.17	0.13	2.37	11.7

Results clearly show that DAF values vary considerably from one bridge to another. Mean DAF values range between 1.03 and 1.23, whereas the maximum DAFs do not fall below 1.14 and can be as high as 2.39. It is important to note that most of the maximum values occurred due to lighter vehicles. Only on bridges SI02 and SI03 were they caused by vehicles that were loaded close to, but still below the legal limits.

Examples of DAF calculation

Figure 4.10 shows the bridge response resulting in the highest DAF of 2.39, measured on any of the test sites. It was caused by a relatively light 20.4 t truck with axle groups 1-2-2 (FHWA Class 9), passing over the SI08 motorway test site. The resonance effects are clearly visible. The bridge oscillations are still present in the signal eight seconds after the vehicle has left the bridge, persisting through the passage of a 1.5 t car at around seven seconds.

Figure 4.10 Event with the highest measured DAF among all vehicles from all sites

Figure 4.11 shows responses for two vehicles crossing a bridge at site SI02. Figure 4.11(a) shows the highest DAF identified on this site. A 30 tonne 5-axle tractor semi-trailer with axle configuration, 1-2-2 (FHWA Class 9) induces a DAF of 2.31. (It should be noted that the maximum total corresponds to the 2nd tandem while the maximum static corresponds to the 1st tandem – an even higher DAF would have been recorded if the maximum static were taken from the 2nd tandem peak). The resonance effect can clearly be seen as the trailer passes and may have been caused by the time between axle arrivals coming close to the 1st natural period of the bridge. The oscillation is sinusoidal around the static response, indicating a simple coupled motion. Figure 4.11(b) shows the bridge response to the passage of an empty 14.1 tonne tractor semi-trailer that resulted in a similarly large DAF of 2.26. In this case, the non-sinusoidal oscillation around the central values indicates a more complex vehicle-bridge interaction.

(a) (b)

Figure 4.11 – Vehicle crossings with the highest DAFs from site si04: (a) Vehicle A; (b) Vehicle B

Figure 4.12 shows the response of the same bridge to the crossing of the heaviest recorded vehicle on this site. The DLA is less than 4 %, much less than in the examples of Figure 4.11. This further supports the extensive evidence that heavier vehicles tend to generate less dynamic amplification.

Figure 4.12 – Event with highest GVW from site SI02

There is further evidence of the trend of reducing DAF for heavier vehicles in site SI07. This site has almost two years of data, with around 750,000 recorded vehicle crossings retained for the DAF calculation, including a number of exceptional heavy transports. With only 6.6-m long RC slab superstructure this bridge was much less prone to dynamic excitation, which resulted in lower DAF values than on site SI02. Figure 4.13 shows the bridge responses for the crossing resulting in the highest DAF value and the crossing by the heaviest vehicle. The highest DAF of 1.29, shown in Figure 4.13(a), is caused by an empty tractor-trailer with axle configuration 1-2-2. In contrast, the permit vehicle travelling at 72 km/h whose response is shown in Figure 4.13(b), is a 110.9 tonne low-loader with 11 axles that has a DAF of just 1.01.

(a) (b)

Figure 4.13 Extreme crossing events from site SI01: (a) event with highest DAF; (b) event with highest GVW

Decrease in DAF with increasing GVW

One of the key findings from the literature and some recent codes is that DAF decreases with increasing load (Caprani 2013, González et al 2009, Kirkegaard et al 1997, Kalin et al 2015 & 2020). To examine the evolution of DAF with increasing loading, Kalin et al (2020) introduce a parameter, DAF_P, the mean DAF from a subset of vehicles, whose GVW is above a cut-off point at the P^{th} percentile of the GVW cumulative distribution. For example, DAF_{10} is the mean DAF of the heaviest 90% of vehicles, i.e., by discarding the lightest 10% of all vehicles. Similarly DAF_{50} is the mean DAF of those vehicles whose GVW is above the median weight. DAF_0 is thus the mean DAF of all the population.

The cut-off points, for the analysis, were chosen at the 0th, 10th, 20th … 90th, plus the 95th percentile. The value of DAF_{99}, calculated from the subset consisting of the heaviest 1 % of vehicles, was calculated only for the four populations that numbered at least 100,000 vehicles. The grouping by percentiles of GVW, rather than by GVW themselves, allows the comparison of evolutions with GVW from all seventeen datasets with substantially different GVW ranges.

As an example, Figure 4.14 shows the scatter graph of DAF against GVW for site SI10. Each point presents a DAF value corresponding to a loading event, either from a single or from several vehicles present on the bridge simultaneously. For multiple presence events, one DAF is calculated and is associated with each contributing vehicle's GVW in the figure. All permit vehicles with GVW above 80 t and most vehicles with GVW above 60 t have DAF values lower than the mean DAF_0 = 1.05. The heaviest exceptional transport, weighing 113 t, had a DAF of only 1.013, making the dynamic increment of this extreme loading case similar to the load effect of a car. The decreasing DAF_P trace demonstrates the trend of decreasing mean DAF with increasing GVW.

Figure 4.14 – DAF values for site SI04

Figure 4.15 summarizes the DAF_P results for all seventeen datasets, grouped into three sets: Slovenian state roads and motorways and US highways. To allow for the datasets with different numbers of vehicles and different GVW distributions, the abscissa gives GVW in percentiles of the maximum. It can be seen for all three datasets that a) there is a consistent decreasing trends of DAF with GVW and b) there is significant variation in the results, illustrated in the figure with the highest

DAF_P values at Site SI01 and the lowest ones at Site SI09. In particular, Kalin et al (2020) found that at the 35 tonne gross weight level, roughly matching fully-loaded large vehicles, either 5-axle semi-trailers in Europe or 18-wheelers in the US, the DAF_P values varied from 1.01 on the Slovenian motorway bridge SI09, to over 1.13 on the American highway bridge, US05.

Figure 4.15 DAF_P values as a function of GVW percentile for three groups of bridges (SI = Slovenia, US = United States)

To elaborate on the variability of the results, Figure 4.16 provides details on the four Slovenian motorway bridges. The results differ significantly, even between the two, almost identical, beam-and-slab twin bridges, S108 and S109. The measured first natural frequencies of both of these bridges were similar, 3.5 Hz and 3.3 Hz, and close to the typical whole vehicle body bounce frequency (Cebon 1999). The main difference was the expansion joint just before the instrumented span of SI08, which excited vehicles' body bounce modes of vibrations just before arrival on the bridge. Its twin bridge, SI09, is 0.6 m longer and structurally identical, but with no expansion joint before the instrumented span, it does not exhibit similar resonance behavior and the DAF_P values are much less.

Figure 4.16 DAF_P values for the four Slovenian motorway bridges

4.5 Conclusions

Design codes prescribe relatively large (conservative) dynamic allowance factors. Conservatism is not expensive for a new bridge and provides some extra reserves of capacity for the structure that may be useful in the later stages of its life. In contrast, for existing bridges, it can be important to obtain realistic measures of dynamics as input to a detailed traffic loading model; this can make the difference between rating the bridge as safe or unsafe. Further work is needed on methods of assessing dynamics in bridges but the evidence strongly suggests that dynamic effects in extreme cases of loading are quite small and much smaller than is allowed for in the major bridge standards.

Two main methods of measuring dynamics are described in this chapter, DAF and ADR. Of these, ADR is the more appropriate because, by definition, it is the ratio of what is required (maximum total LE) to that provided by a static load model (maximum static LE). However, deriving ADR is more complex, requiring a statistical analysis of the data. The concept of DAF is simpler and it has value but, given the evidence of reducing dynamics with increasing LE, it should be treated with caution and some allowance should be made for this reducing trend. Furthermore, field measurements of DAF are necessarily biased and should not be used directly for assessment to avoid undue conservatism.

Quantifying ADR or DAF remains a challenge. Simulations allow great quantities of scenarios to be considered but are only as good as the underlying assumptions on, for example, vehicle and bridge properties. Measurement can address this issue but there is a challenge with quantifying the static response. It is not practical to have vast quantities of vehicles and combinations of vehicles travel over the bridge at crawl speed. Recent Bridge Weigh-in-Motion data has provided large quantities of data but it should be treated with caution until further corroborating evidence becomes available to confirm the accuracy of the static estimates. Nevertheless, the data from Bridge WIM installations is extremely useful. It should be noted that Bridge WIM systems could also collect the data necessary to calculate ADR.