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Characteristic Dynamic Increment for Extreme Traffic Loading Events on Short and Medium Span Highway Bridges

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Title

Characteristic dynamic increment for extreme traffic loading events on short and medium span highway bridges

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Abstract

More accurate assessment of safety can prevent unnecessary repair or replacement of existing bridges which in turn can result in great cost savings at network level. The allowance for dynamics is a significant component of traffic loading in many bridges and is often unnecessarily conservative. Critical traffic loading scenarios are considered in this paper with a model that allows for vehicle-bridge interaction and takes into account the road surface condition. Characteristic dynamic allowance values are presented for the assessment of mid-span bending moment in a wide range of short to medium span bridges for bi-directional traffic.

Keywords

Assessment Dynamic Ratio, ADR, Vehicle, Bridge, Interaction, VBI, Characteristic, Dynamic, Amplification, DAF.

1 Introduction

Site-specific traffic load modelling has been shown [1] to provide great reductions in characteristic load effects, when compared to deterministic load models found in design or assessment codes. Recent improvements in Weigh-In-Motion (WIM) technology [2] have made this possible, by providing road authorities with large databases of vehicle weights, axle configurations and inter-vehicle gaps.

Even with years of WIM data, combinations of vehicles can occur in the lifetime of a bridge that were not recorded. To comprehensively explore the complete design space of loading scenarios, most researchers simulate many more loading scenarios than measurement would allow and apply statistical approaches to the results. The peaks over threshold approach [3], Rice level-crossing technique [4] and extreme value probability distribution fits [5] have been

used to extrapolate from simulated results to find characteristic maximum loading effects. The variability in results can be significant – all of these processes are essentially extrapolations from data collected over a relatively small time to a very large return period.

In this paper a previously developed traffic load modelling approach [6] is used to generate a sufficiently large simulation, that no extrapolation is needed. This model is carefully designed so that simulated load effects match those calculated directly from WIM data, and allows for the simulation of vehicles heavier than any recorded. Using this model 10,000 years of bidirectional traffic were simulated, making it possible to interpolate the lifetime loading effects of interest for a particular bridge, and avoiding some of the problems encountered when extrapolating from shorter simulations.

Static simulation is used to determine the characteristic maximum static load effects in bridges which generally gives results considerably less conservative than the notional load models specified in design or assessment codes. Dynamic amplification is another source of conservatism in bridge assessment and great savings are possible with a site-specific assessment [7] of this phenomenon. The evaluation of vehicle-bridge dynamics is often [8, 9] studied using the Dynamic Amplification Factor (DAF), defined as the ratio of the total load effect, LE_{Total} , to the static load effect, LE_{Static} , for a particular loading scenario (Eq. (1)). DAF values as high as 4 have been recorded [10]. Related definitions such as Impact Factor (IF) [11, 12], Dynamic Increment (DI) [13] or Dynamic Load Allowance (DLA) [14], are given in the literature.

$$DAF = \frac{LE_{Total}}{LE_{Static}} \quad \text{Eq. (1)}$$

However, DAF fails to recognize the reduced probability of both maxima occurring simultaneously, i.e., dynamic interaction and static extreme, and as a result tends to be unnecessarily conservative. For this reason OBrien et al. [5] propose the use of Assessment Dynamic Ratio (ADR), defined as the ratio of characteristic total, \widetilde{LE}_{Total} , to characteristic static load effect, \widetilde{LE}_{Static} , which, in general, correspond to different loading scenarios (Eq. (2)). This ratio is more appropriate for dynamic assessment since it provides the Engineer with the ratio of what is needed, \widetilde{LE}_{Total} , to what can be found by conventional approaches, \widetilde{LE}_{Static} .

$$ADR = \frac{\widetilde{LE}_{Total}}{\widetilde{LE}_{Static}} \quad \text{Eq. (2)}$$

In this paper, the dynamic interaction between vehicles and bridge is modelled a using 3-dimensional vehicle model [15] traversing a finite element plate bridge model [6], which

takes into account the road surface roughness and characteristics of the truck fleet such as speed, weights and suspension properties.

Bridge design and assessment codes are in many cases conservative to allow for generalisation of bridge and traffic characteristics at a safe level [16]. The Eurocode [17] for the design of new bridges is based on a 50-year design life and a probability of exceedance of 5 % in that life which approximates to a return period of 1000-years. The AASHTO design code is based on the expected (mean) 75-year maximum. The HL 93 notional live load model consists of a truck plus a uniformly distributed load, and a dynamic allowance of 0.33 is added to the truck load only. This is intended to reflect the fact that the dynamic effect decreases when more than one truck is on the bridge [18]. For assessment purposes and with site-specific knowledge of loading, much reduced return periods are proposed. Nowak et al. [19] proposes 5 years for the United States. In Europe the authors of the ARCHES report [6] propose 50 years, i.e., 10 % probability of exceedance in 5 years.

In this paper, 5, 50, 75 and 1000 year return periods are all considered. The characteristic ADR is calculated for mid-span moment in each case. This is repeated for a range of short to medium span (7.5, 15, 25, 35 and 45 m) simply supported bridges.

The condition of the road profile is a major factor influencing the response of the bridge to a passing vehicle [7] as well as the specific location of particular bumps [20]. Frequently there is a significant discontinuity in the road profile at the expansion joint, either because expansion joints are susceptible to damage [21], or because of differential settlements of the foundations. To account for damaged expansion joints, the study was repeated including 20 mm deep depressions close to the bridge supports.

1.1 WIM data

As part of the ARCHES project, extensive WIM measurements were collected at five European sites, in the Netherlands, Slovakia, the Czech Republic, Slovenia and Poland. Average daily truck traffic (ADTT) in one direction ranged from 1,100 at the site in Slovakia to 7,100 in the Netherlands. Extremely heavy trucks were recorded at all sites, with the maximum gross vehicle weight (GVW) measured ranging from 106 t at the site in Poland to 166 t in the Netherlands. At the site in the Czech Republic, data on almost 730,000 trucks were collected over a one-year period for two same-direction lanes near Sedlice on the D1/E50 highway between Brno and Prague. At this site the ADTT is 4,751 and the maximum GVW measured was 129 t. This traffic is used as the basis for the work presented here, and a summary of the data collected is given in Table 1.

Total trucks	729,929	
Time period	23 May '07 to 10 May '08	
No. of days with data	235	
No. of "OK days" (weekdays with full record)	148	
Maximum number of axles	11	
Time stamp resolution (sec)	0.1	
	Lane 1	Lane 2
	(slow lane)	(fast lane)
Total trucks	684,345	45,584
Trucks per day on OK Days	4,490	261
Peak average hourly flow on OK Days	242	16
Maximum GVW (t)	129.0	128.0
Average GVW (t)	20.9	17.5
No. over 60 t	322	54
No. over 70 t	149	20
No. over 80 t	61	5
No. over 100 t	10	2
Average speed (km h ⁻¹)	88.2	95.4

Table 1. Overview of WIM data for the Czech Republic

2 Static simulations

The first stage in this study is to estimate the maximum static lifetime bridge loading resulting from the traffic at this site. Various methods have been used in the past to estimate lifetime loading from measured data. In the development of U.S. and Canadian codes for bridge design, Nowak [22, 23] used measurements for a total of 9,250 trucks. Load effects were calculated for these trucks for different bridge spans and plotted on Normal probability paper. The curves were extrapolated to give estimates for the mean 75-year load effect. In the development of the Eurocode [17], traffic measurements were collected over some weeks at different times, and a number of different extrapolation techniques were applied to both measured data and to results from the simulation of a number of years of traffic. The approach used here is to use Monte Carlo simulation, based on the measured data, to model 10,000 years of traffic. Simulated annual maximum load effects are then used to estimate load effects with return periods of 5, 50, 75 and 1000 years. This is done by interpolation using a Weibull extreme value distribution fitted to the annual maxima.

A detailed description of the methodology adopted is given by Enright and OBrien [24], and is summarised here. For Monte Carlo simulation, it is necessary to use a set of statistical distributions based on observed data for each of the random variables being modelled. For gross vehicle weight and vehicle class (as defined here by the number of axles on the vehicle), a semi-parametric approach is used as described in [25]. This involves using a bivariate empirical frequency distribution in the regions where there are sufficient data points. Above a certain GVW threshold value, the tail of a bivariate Normal distribution is fitted to the observed frequencies, and this allows vehicles to be simulated that may be heavier than, and have more axles than, any measured vehicle.

Bridge load effects for the spans considered here are sensitive to wheelbase and axle layout. Within each vehicle class, empirical distributions are used for the maximum axle spacing for each GVW range. Axle spacings other than the maximum are less critical and trimodal Normal distributions are used to select representative values. The proportion of the GVW carried by each individual axle is simulated in this work using bimodal Normal distributions fitted to the observed data for each axle for each vehicle class. The correlation matrix is calculated for the proportions of the load carried by adjacent and non-adjacent axles for each vehicle class, and this matrix is used in the simulation using the technique described by Iman and Conover [26].

Traffic flows measured at each site are reproduced in the simulation by fitting Weibull distributions to the daily truck traffic volumes in each lane at each site, and by using hourly flow variations based on the average weekday traffic patterns in each lane. A year's traffic is assumed to consist of 250 weekdays, with the very much lighter weekend and holiday traffic being ignored. This is similar to the approach used by Caprani et al. [27] and Cooper [28]. For same-lane multi-truck bridge loading events it is important to accurately model the gaps between trucks, and the method used here is based on [29]. The observed gap distributions up to 4 seconds are modelled using quadratic curves for different flow rates, and a negative exponential distribution is used for larger gaps.

The traffic modelled here is bidirectional, with one lane in each direction, and independent streams of traffic are generated for each direction. In simulation, billions of loading events are analysed, and for efficiency of computation it is necessary to use a reasonably simple model for transverse load distribution on two-lane bridges. This is achieved by calculating load effects for each vehicle based on a simple beam, and multiplying these load effects by a lane factor to account for transverse distribution. The lane factors used are based on finite element analyses which were performed on bridges with different spans (from 12 to 45 m), and different construction methods (solid slab for shorter spans, and beam-and-slab for longer

spans). One lane is identified as the “primary” lane and the lane factor for vehicles in this lane is always taken as 1.0. When a vehicle is also present in the other “secondary” lane, the location of maximum stress is identified in the finite element model, and the relative contribution of each truck is calculated. In some cases the maximum stress occurs in a central beam, and the contribution from each truck is similar, giving a lane factor close to 1.0 for the secondary lane. In other cases, the maximum stress occurs in a beam under the primary lane, and the lane factor for the secondary lane is significantly reduced, in some cases to as low as 0.45. Both extremes are modelled in the simulation runs for mid-span moment in a simply supported bridge – with a value of 0.45 representing low lateral distribution and a value of 1.0 representing high lateral distribution.

At the WIM site in the Czech Republic, the data are for two same-direction lanes, and for the purposes of this study, the truck volumes in the faster lane are merged with the slow lane to give a stream of single-lane traffic, with gap distributions adjusted for the slightly higher flow rate. This is similar to the approach that has been used in other studies [30] and is conservative as it neglects the increased gaps between trucks that would be introduced by merging all traffic – trucks and cars – in both lanes. According to Rogers [31], the peak capacity of a two-lane bidirectional road is approximately 2,000 vehicles per hour in each lane, and while the percentage of trucks is site dependent it would typically be in the range of 5 % to 15 % (100 to 300 trucks per hour). The peak hourly flow at this site, 258 trucks per hour, is within this range.

Optimization of the simulation process is achieved through careful program design in C++, parallel processing, and by the use of importance sampling. Parallel processes generate simulated traffic in each lane, while other processes calculate load effects and gather periodic maxima for all event types on bridges of different spans. Importance sampling reduces the amount of calculation by ignoring individual trucks and groups of trucks where the combined GVW is less than some chosen span-dependent threshold (for example 40 t on a 15 m bridge).

The simulation model is calibrated by comparing simulated daily maximum load effects with those calculated for measured traffic. This is done for different loading events, where the event type is defined by the number of trucks present on the bridge when the maximum load effect occurs. The results can be plotted on Gumbel paper, which is a re-scaled cumulative probability distribution. The maximum load effects are sorted in ascending order and plotted against the associated empirical probability. The load effect is plotted on the x -axis, and the y -axis position for value i in the sorted list of N values is given by:

$$y = -\ln \left[-\ln \left(\frac{i}{N+1} \right) \right] \quad \text{Eq. (3)}$$

An example of the comparison between the simulated and observed daily maxima is given in Fig. 1.

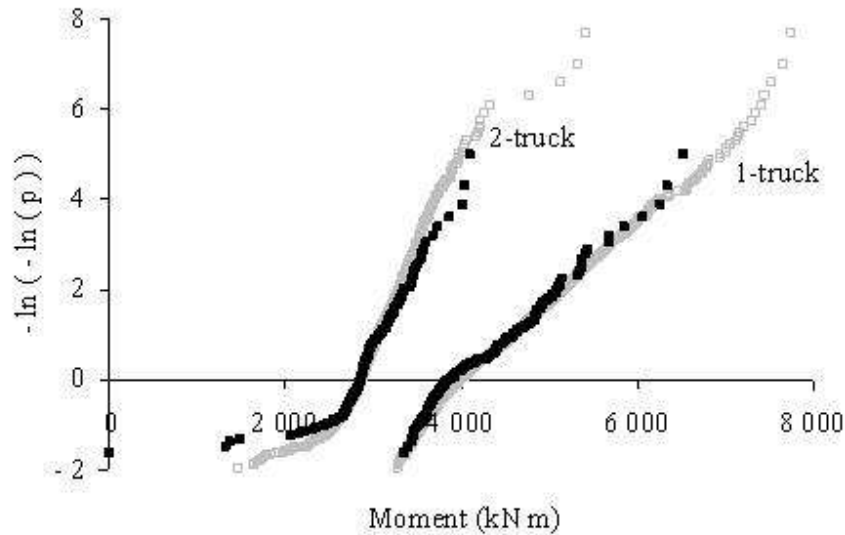


Fig. 1. Daily maximum mid-span moment, for 35 m simply supported bridge and Czech Republic traffic; (•) Observed; (○) Simulated

3 Dynamic simulations

3.1 Vehicle-Bridge interaction model

3.1.1 Vehicle

The response of each individual vehicle is modelled with a 3 dimensional vehicle that consists of two major bodies, tractor and trailer, represented as lumped masses, as illustrated in Fig. 2. The masses are joined to the road or bridge surface by spring-dashpot systems which simulate the suspension and tyre responses. Each axle is represented as a rigid bar with lumped masses that correspond to the combined masses of the wheel and suspension assemblies.

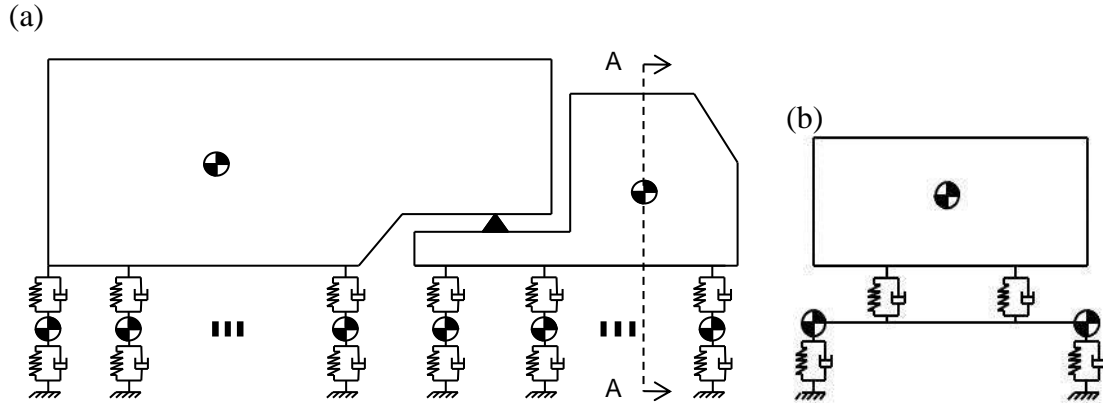


Fig. 2. General vehicle model sketch, (a) Side view, (b) Section A-A

The equations of motion are described in detail in [15] and allow for the definition of a variable number of axles for both the tractor and the trailer. Using the same formulation it is possible to describe articulated trucks, low loaders and crane type vehicles. The vehicle model assumes constant speed, tyre-ground contact at one single point, vertical vehicle forces and linear stiffness and damping elements. Similar vehicle models are widely used in the literature [32 - 34] and are considered to accurately represent vehicle-infrastructure interaction [35].

The model parameters for articulated trucks and crane type vehicles are given in Tables 2 and 3 respectively, together with their corresponding statistical variability used in the Monte Carlo simulations. Longitudinal wheel spacing, axle load distribution and speed are specific to each particular critical loading event obtained from the static traffic model described in section 2. A transverse spacing between wheels of 2 m is assumed. Additional parameters and further comments on selected vehicle parameters can be found in [36].

	Mean value	Standard deviation	Minimum	Maximum	Unit	Reference
Tractor sprung mass	7000	1000	5000	9000	kg	[37]
Steer axle mass	700	100	500	1000	kg	
Drive axle mass	1000	150	700	1300	kg	[34,37,38]
Trailer axles masses	800	100	600	1000	kg	
Steer suspension stiffness	300×10^3	70×10^3	150×10^3	500×10^3	N m^{-1}	
Drive suspension stiffness (air)	500×10^3	50×10^3	300×10^3	600×10^3	N m^{-1}	
Drive suspension stiffness (steel)	1×10^6	300×10^3	600×10^3	1.5×10^6	N m^{-1}	[38]
Trailer suspension stiffness (air)	400×10^3	100×10^3	250×10^3	600×10^3	N m^{-1}	
Trailer suspension stiffness (steel)	1.25×10^6	200×10^3	1×10^6	1.5×10^6	N m^{-1}	
Suspension viscous damping	5×10^3	2×10^3	3×10^3	10×10^3	N s m^{-1}	[34]
Tyre stiffness	750×10^3	200×10^3	500×10^3	150×10^3	N m^{-1}	[37,39]
Tyre damping	3×10^3	1×10^3	2×10^3	10×10^3	N s m^{-1}	[34]

Table 2. Articulated truck parameters

	Mean value	Standard deviation	Minimum	Maximum	Unit	Reference
Axle mass	700	300	500	1000	kg	[34]
Suspension stiffness	4×10^6	80×10^6	3×10^6	160×10^6	N m^{-1}	[40]
Suspension damping	20×10^3	7.5×10^3	15×10^3	30×10^3	N s m^{-1}	
Tyre stiffness	1×10^6	500×10^3	700×10^3	1.8×10^3	N m^{-1}	[39,41]
Tyre damping	5×10^3	3×10^3	2×10^3	10×10^3	N s m^{-1}	[41]

Table 3. Crane type vehicle parameters

3.1.2 Bridge

The bridge is modelled as a simply supported orthotropic thin plate following Kirchhoff's plate theory [42] using the finite element technique [43] for rectangular C_1 plate elements with four nodes [6]. The element has four degrees of freedom at each node, namely one vertical displacement, two rotations and one 'nodal twist' [44], adding up to 16 degrees of freedom per element. Compared to the standard Kirchhoff plate element [42], this element contains one additional degree of freedom per node, included to prevent discontinuity of slope along the edge of the elements. Further information about this plate element and the derivation of the system matrices are given by [6].

Five different concrete bridges have been modelled in this paper with the properties listed in Table 4. The width (11.3 m), Young's modulus in the longitudinal direction ($35 \times 10^9 \text{ N m}^{-2}$), Poisson's ratio (0.2) and structural damping (3 %) are the same for all of them. Although it is known that experimental results suggest a change in damping coefficients with frequency [45], modal damping is used [46] which applies the same dissipation to all modes of vibration.

Span (m)	Thickness (m)	Density (kg m^{-3})	Transverse Young's Modulus (N m^{-2})	1 st Longitudinal natural frequency (Hz)	1 st Torsional natural frequency (Hz)
7.5	0.45	2400	35×10^9	14.02	19.06
15	0.85	2400	35×10^9	6.59	13.92
25	1.40	1800	14×10^9	4.40	12.02
35	1.80	1400	12.5×10^9	3.24	11.66
45	2.20	1000	11×10^9	2.80	11.34

Table 4. Bridge models properties

The properties for the shorter spans (7.5 and 15 m) were chosen assuming solid slabs, made of in-situ or a combination of precast inverted T beams and in-situ concrete [47]. On the other hand, the longer spans (25, 35 and 45 m) are assumed to be of beam and slab construction. They are modelled here as orthotropic plates with higher stiffness in the longitudinal direction than transversely. The fundamental frequencies that arise from these properties are consistent with those recorded by others in field measurements [14, 13, 48].

The road profile is generated as a stochastic process described by power spectral density functions as specified in the ISO recommendations [49] together with the inverse fast Fourier transform method described in [50], which provides realistic road inputs for numerical vehicle

models (Fig. 3). Road classes ‘A’ (‘very good’) and ‘B’ (‘good’) are considered in this paper. The generated profiles are passed through a moving average filter over 240 mm [37] to allow for the width of the tyre contact patch. Vehicles were required to travel a minimum of 100 m on the generated road profile before arriving at the bridge to allow them to reach dynamic equilibrium.

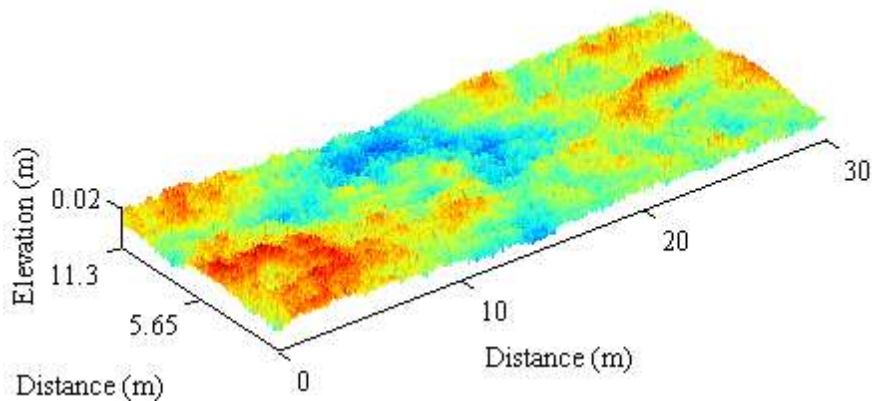


Fig. 3. Example of generated class ‘B’ road profile

The mid-span bending moment in the bridge is found for various vehicles and vehicle combinations, each vehicle moving at a constant speed over the uneven road profile. The vehicle and bridge equations are solved in an iterative procedure [51]. The results were found to agree with results from an experimentally validated 3-dimensional vehicle-bridge-road profile interaction finite element model developed by González et al. [8] and based on Lagrange multipliers using the MSc/NASTRAN software [52].

3.2 Results

The traffic simulations described in section 2 generated 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 was analysed using the vehicle bridge interaction model for two ISO road classes (‘A’ and ‘B’) and two expansion joint conditions (healthy and damaged), adding up a total of 400,000 dynamic analyses. Over 5,000 computer hours were needed to perform the calculations using various dual core processors.

The road profile and vehicles parameters were varied randomly according to the data in Tables 2 and 3 within a Monte Carlo simulation scheme. The bidirectional traffic traversed the bridge as illustrated in Fig. 4.

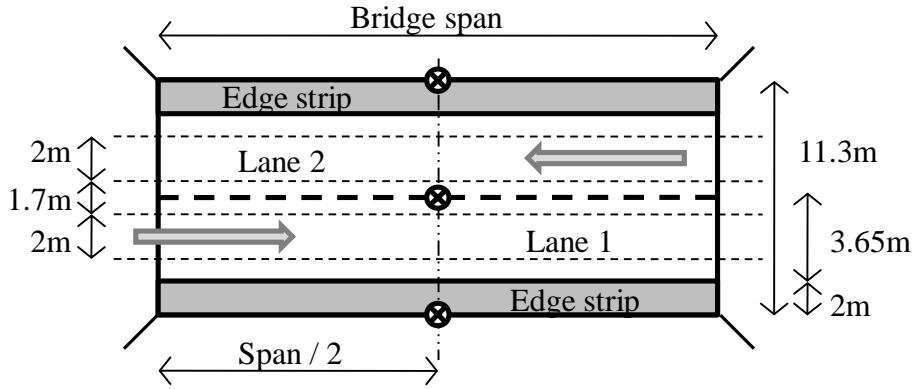


Fig. 4. Sketch of bridge showing wheel paths (--) and points under study (⊗)

Results for a typical example are given in Table 5. For a 45 m bridge, Class ‘A’ profile and ‘high’ lane factor, this table shows the five static and total (static + dynamic) loading events closest to the characteristic load effect for a return period of 50 years. It can be seen that the loading scenarios found to be critical for static load effect are not the same as those that are critical for total. The critical loading scenarios are made up of anything from a single vehicle event to 3 vehicle combinations. (Single and 2-vehicle events are much more dominant for the shorter spans considered). It is also interesting that, even for a sample of five critical events, there is a DAF as high as 1.052 whereas the ADR is only 1.024. In the authors’ opinion, DAF is a very poor indicator of ADR and should not be used to estimate it.

Fig. 5 shows all DAF results for the 45 m bridge, class ‘A’ road profiles and high lane factor (10,000 yearly maximum events). Some DAF values are as high as 1.15 with the higher values tending to occur where bending moment is less. For all span / road / expansion joint / lane factor combinations, the maximum DAF value obtained remained below 1.3.

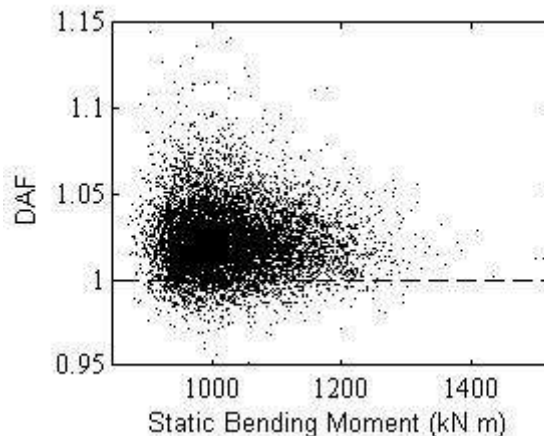


Fig. 5. DAF for 45 m bridge with Class ‘A’ profile, high lane factor

	Rank number	Sketch	Static Moment (kN m)	Total Moment (kN m)	DAF
Static Moment	1		1211	1230	1.016
	2		1211	1251	1.033
	3		1211	1262	1.042
	4		1211	1241	1.025
	5		1211	1217	1.005
Total Moment	1		1200	1239	1.033
	2		1217	1239	1.018
	3		1178	1239	1.052
	4		1194	1238	1.037
	5		1189	1238	1.042
ADR			1211	1239	1.024

Table 5. Top five Static and Total moment loading events for a 50 year return period, for 45 m span, class ‘A’ road profile and high lane factor

3.2.1 Generalized extreme value fits

In this paper the characteristic 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 using maximum likelihood, as proposed in [6]. Other distributions, statistical methods and sampling selection policies have been reported in the literature [3, 4, 53]. However, with such an extensive (simulated) database, this is an interpolation rather than an extrapolation process and the results are insensitive to these assumptions.

When random values are drawn from the same distribution and grouped in blocks of n values, the block maxima will tend asymptotically to the GEV distribution as the block size increases [53]. The GEV cumulative distribution is given by Eq. (4), where μ , σ and ξ are the location, shape and scale parameters respectively:

$$F(u; \mu, \sigma, \varepsilon) = e^{-\left[1 + \varepsilon \left(\frac{x - \mu}{\sigma}\right)^{-\frac{1}{\varepsilon}}\right]} \quad \text{with} \quad 1 + \varepsilon \left(\frac{x - \mu}{\sigma}\right) \geq 0 \quad \text{Eq. (4)}$$

Fig. 6 plots static and total bending moment for the 45m bridge on Gumbel probability paper, together with the GEV tail fits. The studied return periods (5, 50, 75 and 1000 years) are shown.

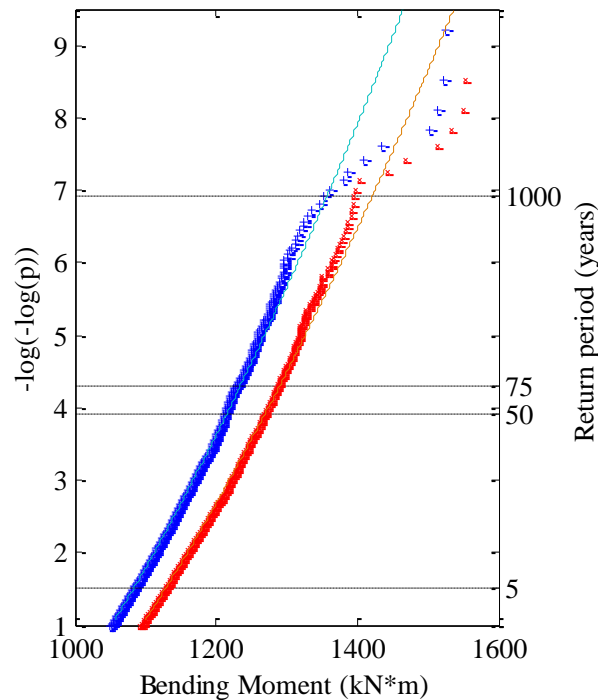


Fig. 6. GEV fits to top 30% of data, Static (+) and Total (x) bending moment on Gumbel probability paper, for 45 m span, high lane factor, Class ‘B’ profile; 5, 50, 75 and 1000 years return periods (--).

As can be seen, the distribution fits show excellent agreement with the data, except, as is expected, for the last 50 or so of the 10,000 points.

3.2.2 Characteristic ADR values

The characteristic ADR values for the analysed return periods are presented in Fig. 7.

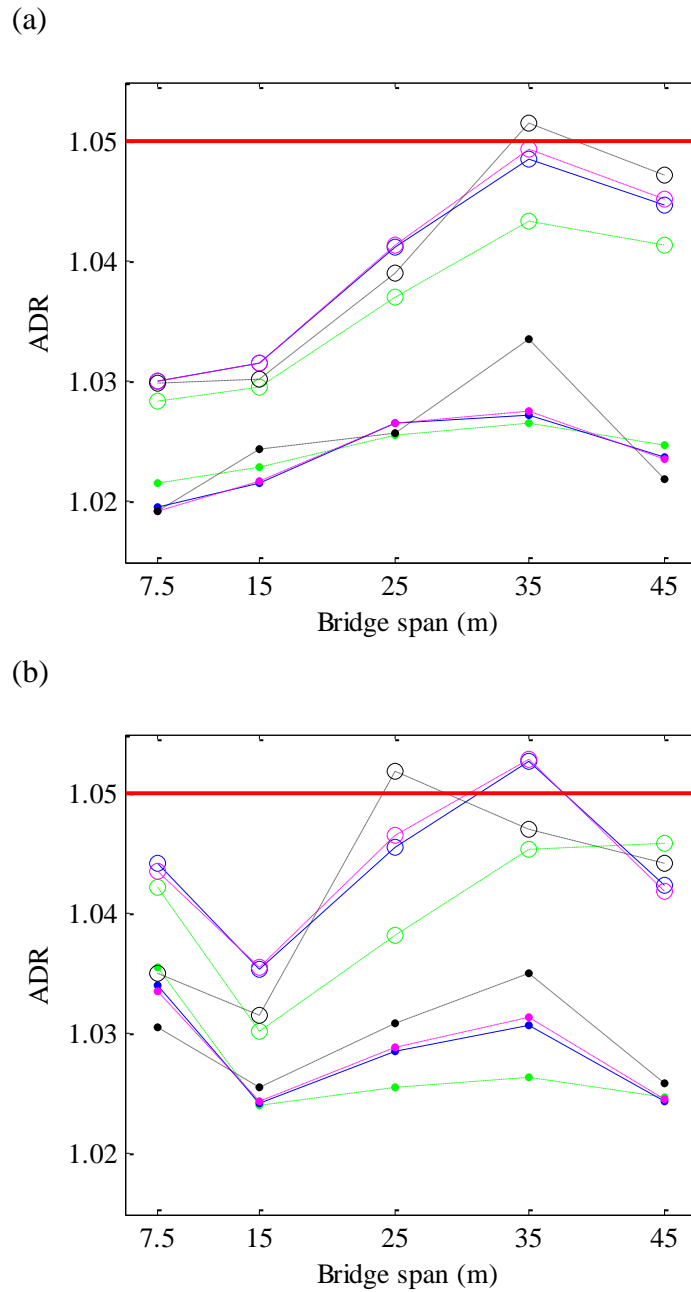


Fig. 7. ADR values for 5 (.....), 50 (—), 75 (-.-.-) and 1000 (-----) year return periods; recommendation (—); for class 'A' (•) and class 'B' (○) profiles; (a) High lane factor; (b) Low lane factor

The results do not present a clear trend, showing a local maximum for the 35 m span. Similar irregular trends have been found by other authors [40, 12]. The differences in ADR values for the different return periods suggest that there is an element of randomness in the results. This

is not surprising given the very small overall magnitudes – for Class A profiles, all ADR values are less than 1.04 (4 % dynamics).

Caprani et al. [27] recommend the separation of the events according to the number of vehicles involved as the underlying statistical distributions are different. This should lead to more accurate distribution fits, needed when long extrapolations are performed. For this paper this approach was tested, and no significant differences were found, since the characteristic values were obtained by interpolation. However, the study of vehicle events by number of vehicles involved did show that 2-vehicle meeting events represent the critical situation for all span / lane factor combinations, except for the 7.5 m span with a low lane factor where single vehicle events produce the characteristic bending moment.

The differences in ADR values between the two road classes are evident, with higher dynamics for greater road roughness. The differences do not show a decreasing trend of dynamic allowance with bridge span, as some codes imply [17, 54]. Again, this may be due to random differences between simulations. Fig. 7 shows that, overall, ADR values are small for the shorter span bridges. Similar results were obtained for the low lane factor.

3.2.3 Influence of bump at expansion joint

It is not unusual to have a discontinuity in road profile at the approach to a bridge associated with damaged expansion joints. For this reason, the analysis was extended to include the influence of such a situation on the characteristic ADR values. The damaged expansion joint was modelled as a 20 mm deep depression over a 300 mm length (Fig. 8(a)), located 500 mm before the centre line of the bearing. The depth of the depression has been chosen following a review of expansion joints surveys on road networks from Japan [55] and Portugal [21].

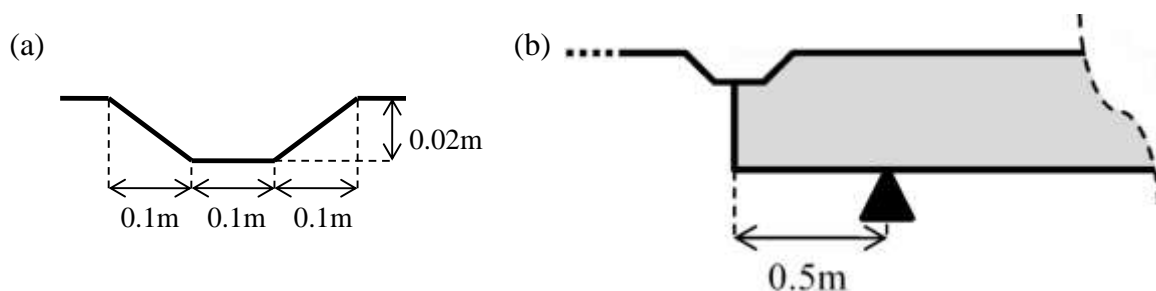


Fig. 8. (a) Damaged expansion joint model. (b) Location of bump with respect to bridge support

ADR values are calculated, as before, by fitting a GEV distribution to the data and interpolating. Results are presented in the form of ‘Bump Dynamic Increment’, DI_{Bump} ,

defined as the difference between the ADR calculated in the presence of the bump and the ADR calculated in its absence. Bump dynamic increments are presented in Fig. 9 for the high lane factor. 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. For larger bridge spans, DI_{Bump} is very close to zero.

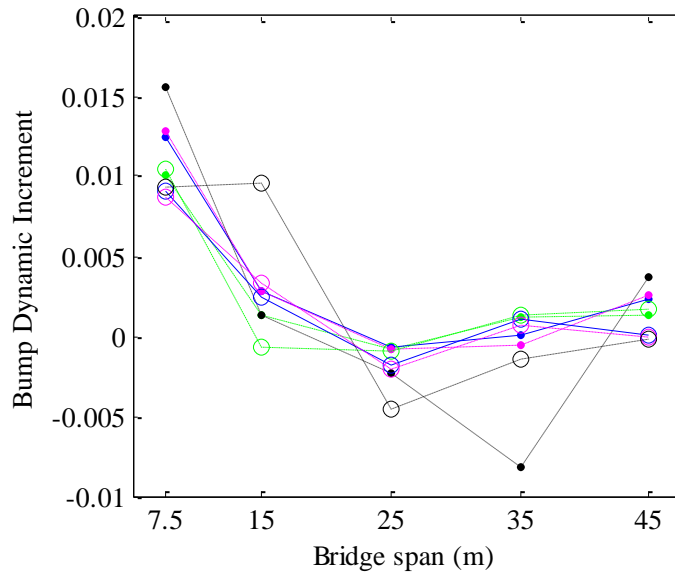


Fig. 9. Bump Dynamic Increment for high lane factor and for 5 (---), 50 (—), 75 (---) and 1000 (.....) year return periods; for class 'A' (•) and class 'B' (○) profiles

A summary of ADR results is given in Table 6 for all spans, profile classes, lane factors and return periods, assuming the presence of damaged expansion joints. In no case does the ADR exceed 1.052, even for Class B road profiles. Given that there are no clear trends with span and that all values are small, it is the authors' opinion that an ADR of 1.05 should be used for all spans in this range where the road profile can be maintained in good condition.

	Return period	Road class	Class A					Class B				
	(years)	Span (m)	7.5	15	25	35	45	7.5	15	25	35	45
High LF	5		1.032	1.024	1.025	1.028	1.026	1.039	1.029	1.036	1.045	1.043
	50		1.032	1.024	1.026	1.027	1.026	1.039	1.034	1.040	1.050	1.045
	75		1.032	1.024	1.026	1.027	1.026	1.039	1.035	1.039	1.050	1.045
	1000		1.035	1.026	1.024	1.025	1.026	1.039	1.040	1.034	1.050	1.047
Low LF	5		1.043	1.025	1.024	1.028	1.026	1.050	1.032	1.036	1.047	1.045
	50		1.042	1.026	1.029	1.031	1.023	1.050	1.036	1.043	1.052	1.046
	75		1.041	1.026	1.029	1.032	1.023	1.050	1.036	1.044	1.052	1.046
	1000		1.037	1.026	1.034	1.035	1.023	1.045	1.033	1.049	1.048	1.047

Table 6. ADR values (LF = Lane Factor)

3.2.4 Mid-span assumption

For various loads over simply supported bridges, it is commonly assumed that the maximum bending moment occurs at mid-span. However, this assumption is not even true for the maximum static moment, which occurs when the centre line of the span is midway between the centre of gravity of the loads and the nearest concentrated load [56, 57]. This effect is magnified when the bridge dynamics are taken into consideration, and significant bending moment differences are obtained between the maximum load effect and the one at mid-span [58].

The dynamic simulations have been checked and the maximum total moment (for any point longitudinally) compared to the maximum mid-span total moment. The parameter, γ is defined as the difference. The average γ values are presented in Fig. 10, averaged over all bump / events combinations. The maximum moment is found to exceed the mid-span moment by 0.01 to 0.02. Hence, if the maximum moment is required, the mid-span ADR of 1.05 recommended above would need to be adjusted to 1.07 (for 7.5 m span) or 1.06 (for all other spans).

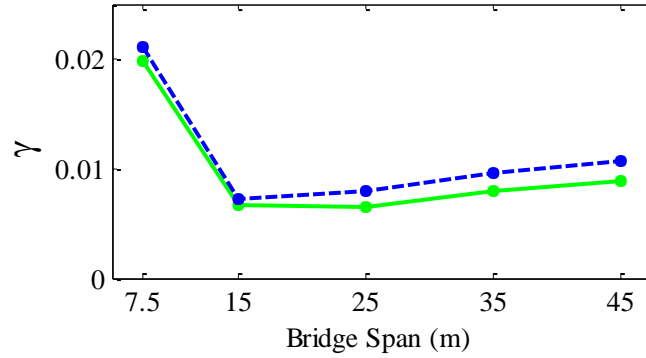


Fig. 10. Average γ values for class 'A' (—) and 'B' (- -) profiles

4 Summary and Conclusions

This paper describes the calculation of lifetime dynamic allowance values, expressed in terms of Assessment Dynamic Ratio, ADR, for a range of short to medium span concrete bridges. 400,000 years worth of traffic loading events were simulated statically for bidirectional traffic and 400,000 annual maximum loading events were simulated dynamically using a 3-dimensional bridge-vehicle interaction finite element model. Monte Carlo simulation allowed for variability in many parameters such as road profile, axle weights and spacings and truck dynamic properties.

It is shown that ADR is generally small. A value of $ADR = 1.05$, is recommended for mid-span moment in simply supported bridges with spans between 7.5 m and 45 m, with well maintained road surfaces (Class 'A' or 'B'). Localised damage at the expansion joint was found to be only important for the shorter spans and is allowed for in the 1.05 recommendation.

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