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Recommendations for dynamic allowance in bridge assessment

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ABSTRACT: This paper summarises the developments in the dynamic response of bridges to traffic loading based on a large amount of simulations and field tests carried out within the 6th EU framework ARCHES (2006-2009). When assessing the total traffic load effect on a bridge, there is a number of alternatives to characterise the associated dynamic component depending on the information available to the engineer. The simplest approach is to adopt the conservative values provided in bridge codes covering for many uncertainties. Nevertheless, if bridge drawings, bridge properties, updated weigh-in-motion data and road profile were known, then validated vehicle-bridge finite element interaction models can be used to reduce these uncertainties. Dynamic allowance can also be experimentally derived from measured total load effects using modern bridge weigh-in-motion technology. Guidelines are provided on how to obtain a site-specific value of dynamic allowance, both numerically and experimentally.

1 INTRODUCTION

1.1 *Code recommendations*

When assessing the dynamic allowance of a bridge due to passing traffic, it is important to take into count the bridge code that was in practise at the time the bridge was designed. If there was no site-specific information available to the engineer, these recommendations represent conservative values to follow. It is common practice to use a Dynamic Amplification Factor (DAF) or a similar parameter such as Dynamic Load Allowances (DLA) (AASHTO 1996) that is applied to the static traffic load. The current Eurocode traffic load model (British Standards 2003) is based on the statistical combination of static traffic load effects and DAFs (Bruls et al 1996). The Eurocode recommends values that are necessarily conservative to cover for an entire range of bridges with different mechanical characteristics, boundary conditions, and the large number of uncertainties associated to the Vehicle-Bridge Interaction (VBI) problem. A more realistic characterisation of the total load effect would require experimental testing and/or the use of complex computer models, and this has been the main objective of the investigations presented in this paper as part of the FP6 European RTD framework project ARCHES (Assessment and Rehabilitation of Central European Highway Structures) (ARCHES 2006-09). The ARCHES project involves partners from Belgium, Croatia, Czech Republic, Ireland, Italy, Poland, Slovenia, Spain, Switzerland and The Netherlands (Wierzbicki 2010).

1.2 *Dynamic amplification in ARCHES*

The uncertainties on dynamic allowance can be reduced by gathering knowledge on the bridge response to the traffic imposed to it. The ARCHES investigation on dynamic allowance is contained in deliverable D10 of Work Package 2 (Casas 2010) and it is structured as shown in the scheme of Figure 1.

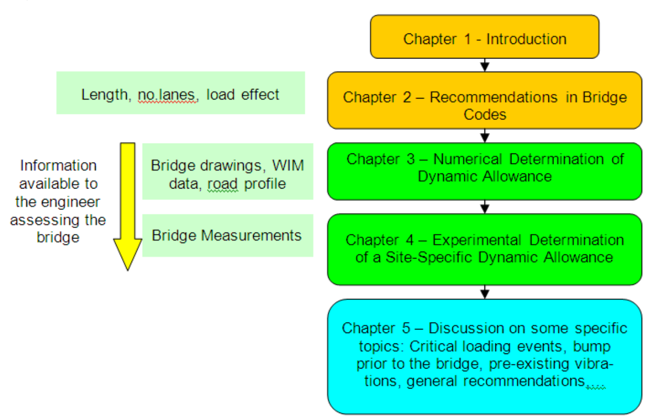

Figure 1. Structure of ARCHES deliverable D10 on dynamic allowance.

Therefore, the D10 report is accompanied by appendices on mathematical models employed for bridges and vehicles, the simulation of their dynamic interaction, and a review of the influence of vehicle, bridge and road profile parameters on dynamic amplification. The most relevant findings are summarised in the sections that follow.

2 DYNAMIC AMPLIFICATION FACTOR FOR CHARACTERISTIC STATIC LOAD EFFECTS

2.1 *Definition of assessment dynamic ratio*

A further level of conservatism in the dynamic allowance specified in design codes occurs due to the independent manner in which critical static load and the corresponding allowance for DAF are specified. Previous research (SAMARIS 2006, González et al 2003), followed by field measurements (Žnidarič & Lavrič 2010, González et al 2010) and VBI Finite Element (FE) simulations in ARCHES (González et al 2009, González et al 2010) show that certain bridges are not susceptible to high levels of VBI when loaded by a 'critically' heavy vehicle or a 'critical' combination of vehicles. In this project, two concepts of dynamic allowance are employed to analyse this phenomena: DAF and Assessment Dynamic Ratio (ADR). DAF is defined here as the ratio of the maximum total load effect to the maximum static load effect caused by the passage of the vehicle or vehicles over a bridge. When referring to DAF, both total and static load effects are related to the same traffic loading event and to the same section in the bridge. ADR is the factor that multiplied by the characteristic static load effect will provide the characteristic total loading effect for a given return period. The characteristic total loading effect and the characteristic static loading effect do not necessarily correspond to the same traffic event.

2.2 *Numerical estimation of dynamic allowance*

Chapter 3 of the deliverable explains how to numerically specify a dynamic allowance when some bridge, traffic and road characteristics are known to the engineer. The procedure is demonstrated for a 32 m long simply supported with two-lanes of traffic running in opposite direction. The bridge is of the beam-and-slab type with 5 longitudinal concrete beams and 5 transverse diaphragms. For bridge assessment purposes, the characteristic static load effect can be found using conventional extrapolation methods: maximum static load effect per day is measured or simulated; the data are fitted to an Extreme Value distribution and extrapolated to find the characteristic static value (more details can be found in Enright et al (2010)). Figure 2 shows the plotted monthly maxima for a typical year. In the figure, the 10 monthly maximum load events for each type of load scenario (1-truck, 2-truck, etc.) are presented. From the observed years of monthly maxima the 10 most critical events overall are extracted (10 worst monthly maxima). As expected for a bridge of this type and span length, the majority of critical events are 1-truck and 2-truck events with occasional 3-truck events contributing.

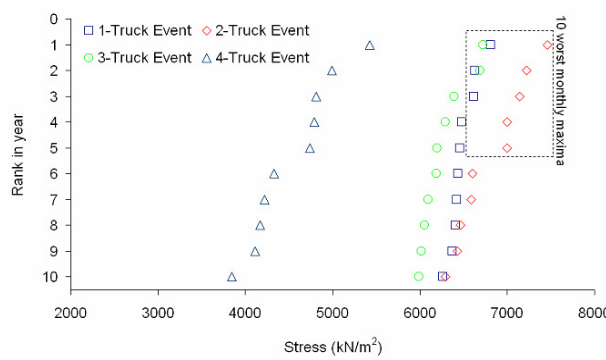

Figure 2. Determination of Worst Monthly Maxima based on Weigh-In-Motion Data

In Figure 3, the total stress and the DAF associated to the worst 100 static loading cases resulting from Monte-Carlo simulations are plotted in the figure. Details on the bridge and vehicle finite element models and the numerical implementation of their interaction can be found in González et al (2008). The mean DAF of these 100 critical loading events is 1.035 with a standard deviation of 0.041. Two relevant conclusions can be extracted from the figure: (1) the variability of the DAF associated to critical loading events is small and (2) the difference between the worst static loading event (ranked 1) and the static loading event ranked 100 is so large that the probability of finding a traffic event outside these top 100 events causing a larger stress becomes negligible. The Eurocode traffic load model has an implicit in-built DAF of 1.17, but in this case, the analysis of the traffic on the site for a particular 10-year return period has led to an ADR of 1.06 (ratio of maximum total to maximum static).

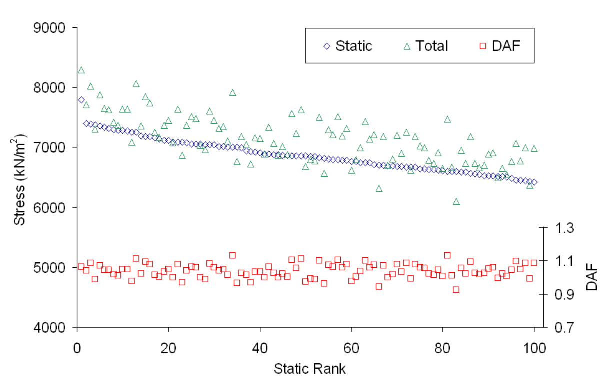

Figure 3. Total stress and DAF versus worst 100 static stress loading cases.

2.3 *Practical implementation of a dynamic allowance*

The ADR value found above faces the problem that in practise it is not possible to take measurements for the return period of the structure. Nevertheless, a relatively short period of time can be used to give a close estimate of ADR (OBrien et al 2009). This is a consequence of previous findings such as the fact that longer measurement periods will lead to traffic events causing larger static responses, and larger static responses are typically associated to smaller % of dynamics (Figure 3). The validity of the approach is tested here with simulations.

So, the response of a bridge is simulated using a one-dimensional numerical VBI model consisting of a single 5-axle vehicle of variables GVW and velocity that traverse a 25 m long simply supported beam. The probability distributions of each variable are defined by WIM data (Figure 4), and are used to obtain the characteristic value for static load effect. Figure 5 shows the result of plotting the static bending moment resulting from sampling the passage of traffic over the bridge from the GVW distribution in Figure 4. Typically, the higher the static strain gets, the smaller the DAF and the variability of DAF become.

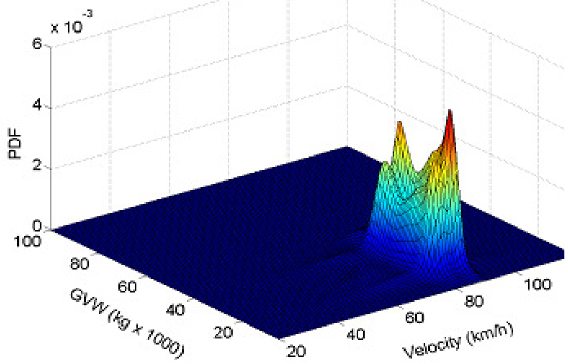

Figure 4. Probability density function of GVW and velocity.

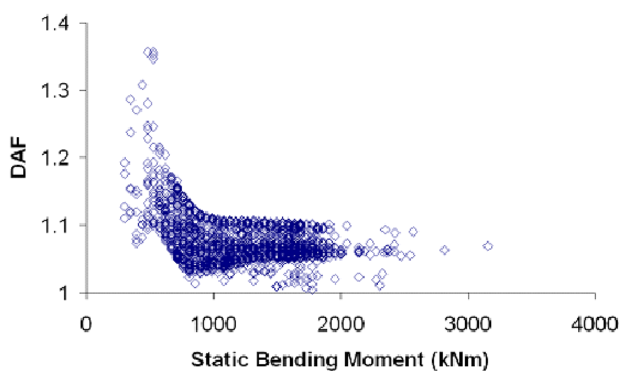

Figure 5. DAF versus static bending moment.

The characteristic total load effect is required for assessment purposes in this theoretical exercise. Similarly, but independently, the distribution of characteristic total load effect is obtained. Then, the cumulative distribution function of maximum static and total bending moments can be generated as shown in Figure 6. Comparison between the total and the static results yields the site-specific allowance for dynamic interaction and a given return period; an ADR of 1.06 results for the 1000-year return period sought in this numerical example (Figure 7).

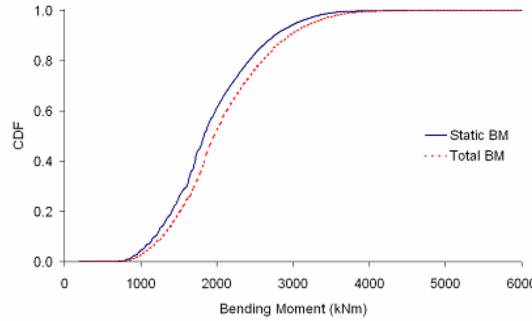

Figure 6. Cumulative distribution function for maximum static and total bending moment.

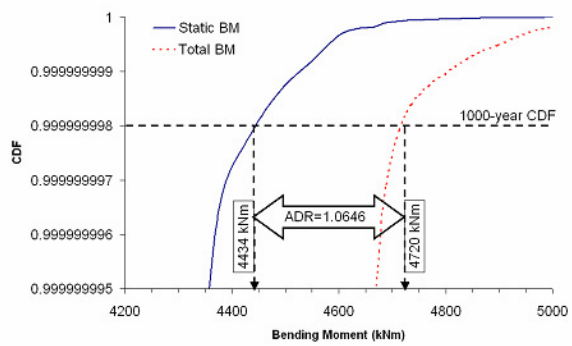

Figure 7. Determination of assessment dynamic ratio.

Figure 8 shows the ADR that results for different return periods. It can be seen how the variability of ADR reduces as the return period increases and how a 1-month ADR provides a reasonable estimate of ADRs with longer return periods.

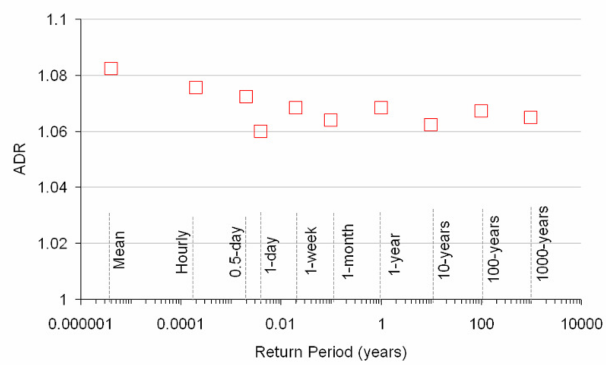

Figure 8. Variation of assessment dynamic ratio with return period.

A further theoretical investigation consisting of a two-truck meeting event, typical of critical loading scenarios, is carried out with an increased number of design variables. In this study both trucks have different GVW and velocity values, with the meeting location of the vehicles also varied. Additionally, 100 different road profiles within class A (‘very good’ according to ISO) are analysed. These are the 100 points appearing along a vertical line for different return periods in the horizontal axis of Figure 9. It can be seen how as the return period increases, the influence of the road profile (or variability of ADR with the profile) decreases.

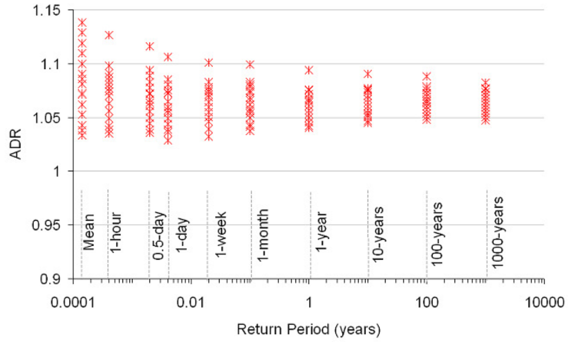

Figure 9. Variation of assessment dynamic ratio with return period and road profile.

2.4 *Experimental determination of dynamic allowance*

Based on the results of the simulations above, D10 proposes a method to calculate an ADR based on strain field measurements. Recent advances in Bridge Weigh-In-Motion technology allow to measure not only vehicle characteristics and weighs from measured strain under the bridge soffit, but also to provide a DAF value for each vehicle event using filtering techniques (Žnidarič et al 2008, Žnidarič & Lavrič 2010). So, the capability of the Si-WIM system to measure maximum total strain and estimate the maximum static strain for each traffic event is used to provide a site-specific recommendation for ADR that can be used in bridge assessment. The dynamics of 5 bridge sites were monitored during the ARCHES project. This section focuses on the analysis of the Vransko bridge in Slovenia. This is a simply supported bridge 24.8 m long (Figure 10). A total of 147524 loading events were recorded at the site between the 25th September 2006 and the 21st November 2006.

Figure 10. Vransko bridge.

In theoretical simulations, the dynamic allowance associated to the critical loading cases are clearly lower than the one associated to light vehicles. This is experimentally verified with measurements of DAF and vehicle weights on site. Figure 11 show the relationship between DAF and maximum static strain for the Vransko bridge. Values in the Eurocode would suggest a higher DAF, but evidence show that for the heaviest vehicles, the maximum DAF does not exceed 1.1 (represented by a horizontal dotted line in Figure 11). The smaller scatter of the tail associated to the maximum static strains of these experimental figures appear to resemble the theoretical results for critical loading cases of Figure .

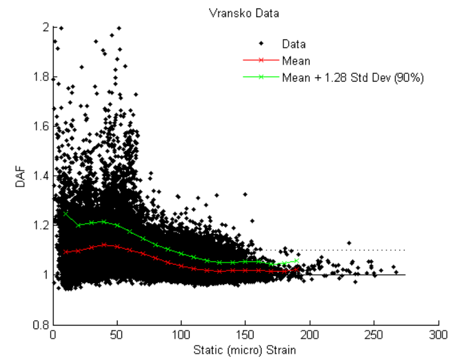

Figure 11. Measured DAF versus static strain.

2-axle and 5-axle trucks were the dominant truck classes and a sample of their DAF distribution is illustrated in Figure 12. The figure below is found in agreement with previous investigations that generally address: (1) Larger DAFs are associated to lighter vehicles (lower modes) and (2) Larger DAFs are associated to vehicles with smaller number of axles.

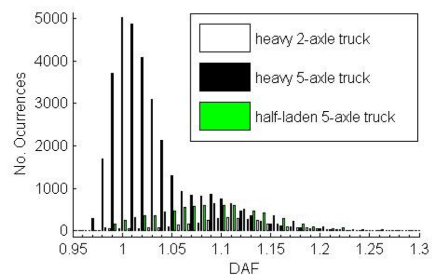

Figure 12. Distribution of DAF by vehicle subclasses.

Figure 13 shows the variation of ADR for three bridges: the Vransko bridge (simple supported 24.8 m long), a bridge in The Netherlands (integral 7.3 m long) and the Trebnje bridge (integral 8 m long) in Slovenia. There is a clear trend for ADR and the variability of ADR to decrease as the sample size increases (as found in theoretical simulations, i.e., Figure 8), except for boundary cases appearing at the extremes (these could be due to outliers, vehicles changing lanes or some kind of interference that corrupted the measurements). Once the sample was large enough, the ADR does not seem to oscillate as much and tends towards a lower bound value. An extended version and a description of other bridge sites can be found out in the deliverable and in González et al (2010).

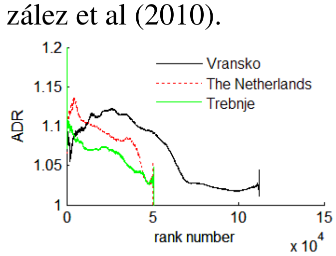

Figure 13. Variation of ADR versus loading event rank number for three bridge sites.

The average DAF values for the extremely heavy vehicles (low-loaders or cranes) and for the multiple presence events with 2 heavy vehicles, was on all 5 measured sites close to 1.The DAF of the heaviest loading events in 3 out of 5 sites was equal to 1.0, and only on a longer simply supported beam-deck bridge, the Vrankso bridge presented here, where the heaviest event was caused by 2 heavy semi-trailers simultaneously on the bridge, the DAF associated to the extreme loading event reached 1.04. Therefore, on that site a considerable bump was measured on the approach to the bridge, which excited VBI.

3 FURTHER INVESTIGATIONS ON DYNAMIC ALLOWANCE

3.1 *Introduction*

D10 reviews a number of special topics concerning dynamic allowance that the engineer should be aware of, since they must lead to a smaller/larger dynamic allowance than anticipated a priori. These topics include the presence of a bump, the existence of pre-existing vibrations, the worst possible load effect when considering all possible bridge sections and differences between dynamic amplification factors due to normal traffic loading (i.e., 5-axle trucks) or exceptional traffic loading (i.e., cranes). A detailed description of the mathematical vehicle models can be found in Cantero et al (2009a). Here the VBI was implemented in Matlab (Cantero et al 2009b) using an iterative procedure as proposed by Green et al (1995) due to its flexibility and computational efficiency to simulate vast amount of traffic events. Results were found in agreement with the alternative VBI approach employed by González et al (2008). The results from these theoretical investigations confirm the low dynamics associated to critical loading events found in field measurements.

3.2 *Influence of pre-existing vibrations*

To assess the influence of bridge vibratory condition prior to heavy loading a number of alternative theoretical road surface profiles were considered. Figure 14 shows the DAF resulting from a 60-tonne single 5-axle articulated vehicle running over a 25 m simply supported bridge and compared to the same vehicle preceded by a 30-tonne vehicle that leaves the bridge in free-vibration. In the figure, the 99% DAF are plotted for a sample of 20 alternative road profiles, each with an IRI of between 1 m/km and 6 m/km. Also shown to aid visualisation are approximate upper bounds for the range of profiles considered. As can be seen the presence of pre-existing bridge vibrations increases the maximum occurring DAF for all profiles considered. The damping of the bridge in the figure is 3% and it plays an important role in the rate at which the pre-existing vibrations decay. Further details can be found in Rattigan et al (2009).

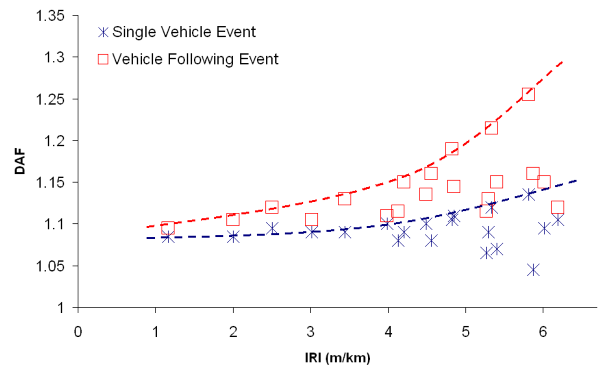

Figure 14. Influence of pre-existing vibrations on dynamic amplification.

3.3 *Differences in total load effect for the critical section and the midspan section*

Most current research on dynamic effects on simply supported bridges focuses on the mid-span section of the bridge, since this location typically holds the worst static bending moment. However, the maximum total moment may be located relatively far apart from the mid-span location and differ considerably from the maximum mid-span moment. Monte-Carlo simulation of the parameters of a 5-axle vehicle model travelling over an Euler-Bernoulli beam were used to analyse this phenomenon. DAF is defined as the maximum total bending moment at midspan divided by the maximum static bending moment at midspan. FDAF is defined here as the maximum total bending moment across the full bridge length divided by the maximum static bending moment at midspan due to the passage of a vehicle. The influence of road profile roughness and bridge length on the magnitude of the differences between mid-span and the worst possible section are summarised in Tables 1 and 2 for ISO (1995) road classes 'A' and 'C' respectively. The tables present the mean values and 95% confidence intervals of 750000 dynamic simulations.

Table 1. Comparison of DAF and FDAF for different bridge lengths and ISO road class 'A'.

Bridge (m)	Mean		95% confidence	
	DAF	FDAF	DAF	FDAF
15	1.020	1.041	1.063	1.084
20	1.021	1.045	1.073	1.098
25	1.023	1.043	1.070	1.092
30	1.029	1.045	1.075	1.090
35	1.032	1.048	1.079	1.095

Table 2. Comparison of DAF and FDAF for different bridge lengths and ISO road class 'C'.

Bridge (m)	Mean DAF	Mean FDAF	95% confidence DAF	95% confidence FDAF
15	1.069	1.110	1.209	1.248
20	1.072	1.116	1.208	1.258
25	1.081	1.130	1.215	1.267
30	1.085	1.137	1.213	1.272
35	1.089	1.135	1.215	1.269

The results above are for a typical European 5-axle truck on a one-dimensional bridge beam model. It was seen that when considering 3-D models and heavier trucks the differences between midspan and the section holding the largest bending moment will tend to be of a smaller magnitude (generally this difference would decrease the higher the static load effect), about a 5% dynamic increment when considering all sections of a bridge with respect to the midspan location. Further details can be found in Cantero et al (2009c).

3.4 *Influence of a bump prior to the bridge*

For short- and medium-span bridges, the road profile appears as a dominant parameter on DAF. The road profile is generally modelled as a stochastic random process. However, this approach does not take into account the high irregularities that are prone to develop in the connection of the bridge to its approach, as result of a damaged expansion joint and/or differential settlement. Planar VBI models are used here to assess the increase in midspan moment and shear effects at the supports that a bump prior to a simply supported bridge may cause. Two types of vehicles were analysed: a 5-axle truck and a 9-axle crane truck. The results in bending moment are summarised in Tables 3 and 4 for the cases of no prior damage and a 4 cm deep expansion joint prior to the bridge. When there was no damaged expansion joint, the cranes may exhibit a slightly higher dynamic component than 5-axle trucks for some span lengths (Table 3).

Table 3. DAF for bending moment at midspan versus span length (no bump prior to the bridge).

Bridge (m)	5-axle truck Mean	5-axle truck 95%	crane Mean	crane 95%
5	1.086	1.176	1.118	1.288
10	1.057	1.128	1.028	1.104
15	1.018	1.055	1.037	1.088
20	1.048	1.092	1.030	1.081
25	1.040	1.087	1.034	1.082
30	1.042	1.086	1.035	1.085
35	1.041	1.079	1.038	1.104
40	1.037	1.074	1.042	1.118

The presence of a damaged expansion joint increases the overall DAF for both type of vehicle configurations, but the 5-axle truck is far more sensitive than the crane, particularly for shorter spans (Table 4). An extended version of the tables (with more span lengths, bump depths and also the analysis of the DAF associated to shear) can be found in ARCHES D10 and González et al (2009).

Table 4. DAF for bending moment at midspan versus span length (4 cms bump prior to the bridge).

Bridge (m)	5-axle truck Mean	5-axle truck 95%	crane Mean	crane 95%
5	1.469	2.139	1.144	1.304
10	1.123	1.280	1.032	1.112
15	1.022	1.066	1.043	1.101
20	1.042	1.087	1.037	1.088
25	1.042	1.089	1.036	1.091
30	1.054	1.102	1.031	1.075
35	1.050	1.092	1.037	1.109
40	1.044	1.085	1.047	1.130

3.5 *Dynamic allowance for exceptionally loaded vehicles (cranes)*

VBI is often considered for the most common classes of vehicle such as the 5-axle articulated truck. However, the dynamic response of bridges to this type of trucks is quite different from the bridge response to the vehicles more likely to feature in maximum-in-lifetime traffic loading events. Large (>100 tonne) cranes and crane-type vehicles that have been recorded at WIM sites in Europe (Cantero et al 2009b, Enright et al 2010). Here, the total bending moment due to these vehicles on short to medium span bridges is compared to 5-axle articulated trucks using 3-D VBI FE models. To account for the variability in vehicle characteristics, more than 40000 VBI events are computed using Monte Carlo simulation based on 77 vehicles (77 worst 5-axle trucks and 77 worst cranes) generating the daily maxima loading effect. Four spans were considered, this is, 7.5, 15, 25 and 35 m. For the 7.5 m bridge, two boundary conditions were analysed: fixed-fixed and simply supported. Variability was allowed in vehicle mechanical properties, speeds and mass distribution, and the results are presented in Table 5. It must be noted that when assessing a bridge close to a fixed-fixed support condition (i.e., an integral type), the DAF values will be significant lower than a simply supported condition or those general recommendations given in bridge codes.

Table 5. DAF for bending moment versus span length (m)

Bridge (m)	Boundary	5-axle truck Mean	5-axle truck 95%	Crane Mean	Crane 95%
7.5*	Fixed-Fixed	1.004	1.047	1.000	1.039
7.5**	Fixed-Fixed	1.000	1.053	0.999	1.047
7.5**	Simply Sup.	1.008	1.063	1.022	1.066
15**	Simply Sup.	1.015	1.087	1.015	1.043
25**	Simply Sup.	1.022	1.101	1.014	1.056

* Moment at the end support. **Midspan moment

It has been observed the scatter of the DAF distribution generally increases for longer bridge spans.

When comparing both types of vehicles, the most frequent DAF values for cranes are smaller than for 5-axle trucks. Therefore, the histograms of DAF versus number of ocurrences are narrower for cranes than for 5-axle trucks. Table 6 shows the equivalent of Table 5 when considering the worst possible section, this is, FDAF. The results show similar standard deviation for DAF and FDAF. Generally, the higher the static loads the smaller the difference between DAF and FDAF becomes as discussed in a previous section. In this table, all FDAF values for cranes remain below 1.1.

Table 6. FDAF for midspan bending moment versus span length (m)

Bridge (m)	Boundary	5-axle truck Mean	5-axle truck 95%	Crane Mean	Crane 95%
7.5	Fixed-Fixed	1.007	1.06	1.009	1.058
7.5	Simply Sup.	1.021	1.079	1.028	1.072
15	Simply Sup.	1.025	1.095	1.016	1.045
25	Simply Sup.	1.058	1.138	1.022	1.063

3.6 *General recommendations*

ARCHES provides general recommendations for assessment of 1-lane and 2-lane bridges (both moment and shear load effects) with ISO road classes ‘A’ and ‘B’ (ISO 1995) that are shown in Figure 15.

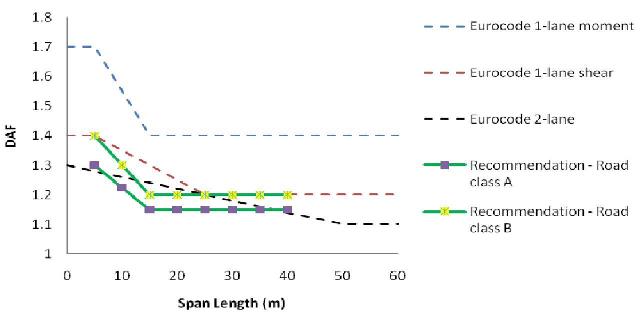

Figure 15 - DAF recommendation versus Eurocode values.

The values provided in the recommendations represent an upper envelope that covers for a large amount of Monte-Carlo simulations varying road profile and traffic static and dynamic properties. These recommendations take into account the maximum total load effect for the entire bridge length (or FDAF). For assessment of 1-lane and 2-lane bridges with a road class ‘A’ (both moment and shear load effects), ARCHES recommends DAF values that varies linearly from 1.3 for a 5 m bridge to 1.15 for a 15 m bridge. Then, DAF remains constant at 1.15. For road class ‘B’, the DAF recommendation also varies linearly between 1.4 and 1.2 from 5 m to 15 m respectively. Then, it remains constant at 1.2. The recommended dynamic allowance represents a significant reduction with respect to the 1-lane values built within the Eurocode traffic load models for both road classes. For 2-lane bridges, the recommended values are also smaller than Eurocode values if the road profile was a class ‘A’.

4 CONCLUSIONS

Site specific assessment of traffic loading has considerable potential to prove that bridges are safe which would otherwise have been rehabilitated or replaced. This is due to the conservatism of bridge standards that cover a wide range of possible traffic loading conditions throughout the road network and uncertainties on how the bridge will react to these conditions. ARCHES deliverable D10 has shown how dynamic allowance for traffic loading on an existing bridge can be determined using validated VBI FE models, if bridge drawings, measured bridge properties and road profile, and updated WIM data for the site were available. Alternatively, the dynamic allowance can be experimentally derived from measured total load effects using modern Bridge-WIM technology. A simpler approach is to adopt the large dynamic allowance given in bridge codes that must cover for the many variables and uncertainties associated to the VBI problem. There is clearly a considerable gap between the complex mathematical modelling and experiments required for an accurate determination of dynamic allowance and the conservative values available at bridge codes. In order to reduce this gap, ARCHES has proposed an intermediate solution based on the large amount of experimental tests and numerical simulations carried out during the project. The proposed envelope of dynamic effects of Figure 15 offers an inexpensive way to give a preliminary realistic assessment of the dynamics of a bridge purely based on its length and the road class.

The quality of the road profile plays a role that becomes more dominant as the span length decreases, but in the case of very good road profiles (ISO class ‘A’), the critical loading cases governing the maximum load effects typically produce dynamic amplification factors below 1.1. Nevertheless, the presence of a bump or a damaged expansion joint prior to the bridge may lead to higher values in short span bridges. Even so, it has been shown that exceptionally heavy vehicles representing critical loading cases such as cranes, have a rigid configuration that generates smaller dynamics than typical 5-axle articulated trucks. So, if the road profile of a bridge was maintained in a good condition, the dynamic amplification factor associated to the critical loading cases could be substantially reduced in relation to the values built within the Eurocode traffic load models.

Finally, further reductions in dynamic allowance can be achieved if a better knowledge of the bridge response was acquired through numerical simulations and field tests. Most probably, measurements will show that DAF is considerably less than what is reflected in Figure 15. In fact, the five bridge measurements on heavily trafficked motorways carried out within the ARCHES project, lasting from 2 weeks to 2 months, on three integral slab bridges, on one simply-supported beam-deck bridge and on one 6-span simply-supported beam bridge with a continuous deck, consistently showed that the DAF decreased as a function of increasing weight of the loading events, and the dynamics associated to the critical loading case was close to 1.0. So, DAF measurements will optimise assessment of the existing bridges, because (a) the measured DAF values will likely be much lower than those prescribed in the design codes and (b) consequently, knowing the real DAF reduces uncertainties of the structural safety assessment which can be employed through lower safety factors for traffic loading.

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