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Characteristic Dynamic Traffic Load Effects in Bridges

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Abstract

When formulating an approach to assess bridge traffic loading with allowance for Vehicle-Bridge Interaction (VBI), a trade-off is necessary between the limited accuracy and computational demands of numerical models and the limited time periods for which experimental data is available. Numerical modelling can simulate sufficient numbers of loading scenarios to determine characteristic total load effects, including an allowance for VBI. However, simulating VBI for years of traffic is computationally expensive, often excessively so. Furthermore, there are a great many uncertainties associated with numerical models such as the road surface profile and the model parameter values (e.g., spring stiffnesses) for the heavy vehicle fleet. On site measurement of total load effect, including the influence of VBI, overcomes many of these uncertainties as measurements are the result of actual loading scenarios as they occur on the bridge. However, it is often impractical to monitor bridges for extended periods of time which raises questions about the accuracy of calculated characteristic load effects.

Soft Load Testing, as opposed to Proof Load or Diagnostic Load Testing, is the direct measurement of load effect in bridges subject to random traffic. This paper considers the influence of measurement period on the accuracy of soft load testing predictions of characteristic load effects, including VBI, for bridges with two lanes of opposing traffic. It concludes that, even for relatively short time periods, the estimates are reasonably accurate and tend to be conservative. Provided the data is representative, Soft Load Testing is shown to be a useful tool for calculating characteristic total load effect.

Keywords

Soft Load Testing, Vehicle Bridge Interaction, VBI, Weigh-In-Motion, WIM, Bridge, Dynamic, Vibration, Characteristic.

1. Introduction

Site specific assessment of traffic loading has considerable potential to prove that bridges are safe which would otherwise have been rehabilitated or replaced [1]. This is because standards are necessarily conservative, representing a wide range of possible traffic loading conditions throughout the road network. For most bridges, standards tend to be particularly conservative in their allowance for dynamic vehicle bridge interaction (VBI). Evidence is emerging [1-3] that VBI reduces as load effect increases – see, for example, Figure 1. If this trend is generally applicable, it suggests quite a small required VBI allowance for large return periods such as 75 or 1000 years. This paper proposes a method of field measurement of the strains in bridges due to VBI, to calculate a characteristic value that can be used in bridge assessment.

Figure 1.

Two of the most common field methods of improving knowledge of bridge load capacity are Proof Load Testing and Diagnostic Load Testing. In Proof Load Testing, the response of a bridge to loading is measured using heavy pre-weighed test vehicle(s) (the ‘proof’ load). The bridge is then deemed to be safe to carry a portion of the proof load.

Diagnostic Load Testing involves driving pre-weighed trucks across an instrumented bridge, often for various transverse locations at both a crawl speed (pseudo-static test), and full highway speed (dynamic test). The results are used to calibrate and validate a numerical model of the bridge/vehicle system which can subsequently be used in numerical simulations.

Soft Load Testing, proposed by Žnidarič [1], offers a low-cost method of improving the accuracy of bridge safety estimates. It is being developed as part of the *ARCHES* research project [4] which involves partners from Belgium, Croatia, Czech Republic, Ireland, Italy, The Netherlands, Poland, Slovenia, Spain and Switzerland, and is part-funded through the European Commission's 6th Framework research programme. Soft Load Testing is based on bridge Weigh-In-Motion (WIM) measurements [5-8], where instrumented bridges are used to determine the static weight of vehicles as they pass overhead. Recent Bridge WIM algorithms include ‘measured’ influence lines [9] which give an accurate relationship between load effect and static vehicle/axle weight.

Using the bridge WIM instrumentation, sometimes with additional sensors, total bridge load effects can be measured including the effects of VBI. Comparing total load effect with the static equivalent gives the influence of VBI, i.e., the dynamic increment.

2. Description of the Model

It is difficult to secure sufficient field data for statistical studies involving years or decades of traffic loading on bridges. Therefore, this paper describes a study based on a numerical model for which the exact solution can be calculated. To simulate dynamic loading scenarios, a numerical VBI model for bridge crossings by single and multiple trucks was developed. To illustrate the phenomenon of soft load testing, a very simple, computationally efficient model is required to allow millions of repeated calculations. Hence, a 1-Dimensional model is chosen and the number of design variables is first reduced to two: gross mass and vehicle velocity. Gross mass is chosen as a variable as it is a primary determinant in defining a critical static load effect while velocity is strongly linked to dynamic interaction [1]. As it is 1-dimensional, this model would not be expected to be very accurate. Similar models were compared to more sophisticated models and to field measurements in a previous study [10,11].

A five-axle articulated vehicle is used with eight independent degrees of freedom: bouncing and pitching motion of the tractor centre of gravity, pitching motion of the semitrailer centre of gravity and vertical hop motions of each axle assembly, as described by Harris et al. [12]. This vehicle model traverses a 25m long simply supported beam. The axle spacings are fixed at 3.1m, 5.1m, 1.1m and 1.1m between the 1st-2nd, 2nd-3rd, 3rd-4th and 4th-5th axles respectively. Similarly the distributions of gross mass to the axles are fixed at 12.7%, 27.7%, 19.86%, 19.86% and 19.86% for axles 1 to 5 respectively.

The model incorporates road irregularities in the form of numerically generated road surface profiles. These 1-dimensional profiles are generated based on the International Standards Organisation's method of representing road surface roughness with a power spectral density function. For single vehicle events, 100 profiles are considered with IRI values in the range $1 \text{ m/km} < \text{IRI} < 6 \text{ m/km}$, ($4 \text{ m/km} < \text{IRI} < 6 \text{ m/km}$ corresponds to "good" [13]).

For vehicle meeting events which are, by their nature, more computationally expensive, 25 profiles are considered. A two-truck meeting event is defined here as a case where two trucks contribute to the maximum load effect. Only two-lane opposing traffic is considered. The probability of occurrence of a two-truck event is then the probability of a truck being present in the second lane when the truck in the first lane is at the critical point on the influence line. This probability is calculated as the portion of time in which a truck is present on the second lane of the bridge. For single vehicle events, 2000 vehicles are considered per day. Applying the probability of the second truck being present to this number of trucks gave 57 vehicle meeting events per day.

A weighted tri-modal Normal distribution is fitted to a measured histogram of velocity obtained from WIM data with parameters as shown in Table 1. More than one mode is used to model the distribution since visual inspection of the data identifies local peaks around 83km/h and 88km/h. O'Brien et al. [14] have shown that a maximum likelihood fit of a trimodal normal distribution to WIM data, while being reasonably representative of the data overall, can be particularly poor in the tail region – Figure 2. They propose instead a "semi-parametric" approach which involves simulating directly from the measured histogram below a threshold, where there is deemed to be enough data to

ensure that the frequencies are representative. This is sometimes known as 'bootstrapping' - the probability of any given weight being simulated is taken as the relative (measured) frequency. Beyond the threshold, where measurements are sparse, they propose fitting a normal tail to the observed frequencies. This semi-parametric approach significantly improves the quality of the fit of a theoretical distribution to the data and is adopted here. A localised drop in frequency is observed at approximately 65 tonnes. Thus one parametric tail is fitted for a range from 50-64 tonnes, and another for 65+ tonnes. Parameters for the tail fit are shown in Table 2.

Figure 2.

3. Assessment Dynamic Ratio (ADR)

Many authors [1,10,11,15] report values of dynamic amplification factor, defined as the ratio of total load effect to the corresponding static load effect. Both the static and the total load effect in such a calculation correspond to the same loading scenario. Bridges are designed and assessed for characteristic load effects. For example, the Eurocode [16] for new bridge design is based on load effects corresponding to a 1000-year return period. For bridge assessment purposes, the characteristic static load effect can be found using conventional extrapolation methods [17-19]: maximum static load effect per day (for example) is measured or simulated; the data are fitted to an Extreme Value distribution and extrapolated to find the characteristic static value. The characteristic total load effect (including VBI) is required for assessment purposes. Hence, the required ratio is the Assessment Dynamic Ratio (ADR) [3], defined as the ratio of characteristic total to characteristic static load effect, i.e., the ratio of what is required to what is available. It is of interest that these two characteristic values may not necessarily arise from the same loading scenario.

For the single vehicle 1-Dimensional example described above, the load effect is a function of the two variables, gross mass and speed. Integrating over all possible combinations of these two values, the cumulative distribution function (CDF) is found for the static and total load effects (Figure 3). It should be noted that, as this is done by integration, it is an exact solution (except for discretization inaccuracies); there are no inaccuracies arising from fitting an Extreme Value distribution to maximum-per-day load effect data. It can be seen that, except for very low bending moment, characteristic total load effect exceeds characteristic static as would be expected. More significantly, the difference between total and static tends to increase as the load effect increases towards ever greater, and rarer, extremes. However, the ratio between the two is actually reducing.

Figure 3 (a) & (b).

For 2000 trucks per day over 250 working days per year over a return period of 1000 years, the characteristic loading event is that with a probability of 1 in $(2000 \times 250 \times 1000)$

=) 500×10^6 . This corresponds to a probability of exceedence of $1 - 1/(500 \times 10^6)$. Figure 3 (b) shows that the characteristic total with this probability of exceedence is 4720 kNm and the corresponding characteristic static value is 4434 kNm, giving an ADR of 1.065.

CDF's of mid-span strains measured on the Hrastnik Bridge in Slovenia are illustrated in Figure 4. This represents 14 days of measurement and a total of 5,276 loading events. Total strain was measured directly using sensors attached to the underside of the bridge. The corresponding static strains were found indirectly: a Bridge WIM system, calibrated using statically pre-weighed trucks, was used to infer the static axle weights from the measured strains. The bridge influence line was then used to calculate the strain corresponding to the static axle weights. It should be noted that the current generation of Bridge WIM systems are typically not better than Class B(10) or B+(7) accuracy [20], where B+(7) means that most gross masses inferred by the WIM system are within 7% of the corresponding statistical weighted mass. This inaccuracy in the static weights will inevitably result in inaccuracies in the calculated static strains.

Figure 4.

While the measurements of Figure 4 are for a different bridge geometry, span and traffic from that used in the numerical simulation of Figure 3, the results still confirm some of the main features of the simulations. For both simulated and measured results, static and total load effects track each other closely with total being greater than static. While the difference between static and total increases for higher load effects in Figure 3, the percentage difference is actually reducing. The differences between static and total are less in Figure 4 than Figure 3 but the same phenomenon is present; that of the percentage differences reducing as load effect increases. The 'kinks' in the measured CDF (Figure 4) may be due to the presence of multiple-presence events among the measurements which will be statistically different from single vehicle crossing events. The simulations are for single vehicle crossing events only.

4. Extrapolation of Measured Load Effects to Obtain Characteristic Values

Many authors [21,22] use Extreme Value theory to obtain site-specific characteristic static load effects. Maximum-per-day (or maximum-per-month etc.) load effects are taken from measurements or simulation. For the example considered here, precise characteristic values have been found by integration so it is possible to illustrate the errors that can arise from the conventional approach. In Figure 5, twenty five simulated daily maxima are fitted to a Generalised Extreme Value (GEV) distribution [21] which is extrapolated to obtain an estimate of the 1000-year return period load effect. This process is repeated ten times and the results compared to the precise value found by integration. The errors, as illustrated, range from -24% to +33% demonstrating that, even for *static* load effects, 5 working weeks (25 days) of data is insufficient to get an accurate estimate of the characteristic value.

Figure 5.

Accuracy improves considerably when more data is available. When 100 maximum-of-5-weeks static load effects are fitted to a GEV in Figure 6, the error in characteristic value is only 3.8%. Similarly, when a GEV is fitted to the 100 5-week maximum *total* load effects, the results are good – just 5.6% error. When these errors are combined, the error in ADR is more significant. For this example, the ADR is 1.065 but the ratio of the values found by fitting to GEV distributions is 1.083. While this corresponds to a 29% error in the dynamic element of ADR (comparing 8.3% to 6.5%) the absolute error is not excessive.

Figure 6.

The model used to derive the 5-week maximum total load effects of Figure 6 is 1-Dimensional, which makes it feasible to simulate ($100 \times 5 \times 5 =$) 2500 working days. To simulate 2500 working days (10 years) of vehicle crossing events dynamically is computationally demanding and becomes impractical for 2-and 3-dimensional models. However, static simulations are extremely fast and an estimate of characteristic total load effect can be found if only the 5-week static maximum loading scenarios are simulated dynamically. Caprani et al. [3] and González et al. [23] used this approach for the Mura River bridge in Slovenia. A 3-dimensional finite element model was calibrated using Diagnostic Load Testing. One hundred 5-week maximum static load effects were found using a simple 1-dimensional model and each of these 100 loading scenarios was analysed dynamically using the 3-dimensional model. Nevertheless, this process is highly demanding computationally.

5. The Variability of ADR with Return Period

It can be seen in Figure 3 that the difference between total and static load effect tends to increase as the probability of exceedence decreases (or as the return period increases). However, the *relative* difference decreases with increasing return period. The same phenomenon has been reported in experimental studies by Žnidarič [1]. Here, it is proposed to exploit this phenomenon by using ADR values from relatively short periods of measurement or simulation as conservative estimates of the ADR's for much more rare events.

5.1 Single-Vehicle Events

The relationship between ADR and return period is initially considered using full 1-dimensional simulation, over all possibilities of gross mass and velocity, of both static and total load effect due to a single-vehicle event. The variation in ADR with return period is illustrated in Figure 7. It is seen that there is some fluctuation in the values but the trend is generally that an ADR measured over a short period such as 1 week or 5 weeks is reasonably accurate and is often conservative.

Figure 7.

This simulation is repeated for the 100 randomly generated road surface profiles discussed in section 2. It can be seen in Figure 8 that the standard deviation of inferred ADR narrows as return period increases. The mean plus 1.64 standard deviations line shown in Figure 8 represents a 95% confidence interval, i.e. only 5% of values will exceed this. With 5 weeks of data, the mean + 1.64 standard deviations gives a reasonably accurate and generally conservative estimate of mean + 1.64 standard deviations of 1000-year ADR. The tendency towards conservatism comes principally from the decrease in standard deviation as return period increases.

Figure 8 (a) & (b).

In practise it is generally not feasible to repeat the calculation of 5-week maxima 100 times so the mean and standard deviation of 5-week ADR will not be accurately known. An indication of the implications of using single 5-week values to estimate the 1000-year values is given in Figure 9. The 5-week ADR's for the same 100 road profiles are plotted in this figure against the exact characteristic 1000-year ADR. The 5-week ADR's are generally conservative estimates (below the diagonal in the figure) and this conservative trend increases as the ADR gets greater, i.e., using 5-week ADR as an estimate of 1000-year ADR tends to be more conservative if the ADR is greater.

Figure 9.

The coefficients of correlation between ADR for each of the considered return periods and the 1000-year ADR are given in Table 3. With only 5 weeks of data, there is already a coefficient of correlation of almost 95% with the 1000 year ADR.

5.2 Vehicle Meeting Events

As bridge loading events become more complex, it is reasonable to expect that ADR decreases. For a single vehicle event, there is a significant probability that some vehicles will travel at a speed which excites the 1st natural frequency of the bridge. However, for a two-vehicle meeting event, constructive interference, where the relative

positions and speeds of the vehicles are such that they both contribute to dynamic amplification, is much less probable.

The 1-dimensional model is used here to simulate 2-vehicle meeting events. For 25 different road profiles, ADR is calculated for a range of return periods as before (Figure 10). As anticipated, ADR is generally less than for single vehicle crossing events.

Figure 10.

Table 3 shows that the coefficient of correlation is less than before – greater variability in the nature of the loading scenarios has the effect of increasing the quantity of data needed to obtain a reliable estimate of 1000-year ADR. Figure 11 shows a similar trend to Figure 9, i.e., for meeting events as well as single vehicle crossings, 5-week ADR tends to give a conservative estimate of 1000-year ADR and the degree of conservatism tends to increase with the magnitude.

Figure 11.

Discussion and Conclusions

In realistic traffic loading scenarios, there is considerably more variability than in the simulations considered here. There is a great variation in vehicle classes, axle spacings, articulations, weight distribution between axles and suspension stiffnesses. As this variability increases over the simple examples considered here, it seems likely that the variability in 5-week ADR will also increase. Nevertheless, in the absence of an alternative method of estimating 1000-year ADR, an on-site measurement of the 5-week value is a reasonably accurate and generally conservative estimate. To address the variability issue, the measurement needs to be repeated a number of times. For example, with 25 or 50 weeks of measurement, i.e., five or ten 5-week ADR's, the mean plus 1.64 standard deviations should give a good and generally conservative estimate of the mean plus 1.64 standard deviations of 1000-year ADR. Using the standard deviation of 5-week ADR captures the variability in the results and, as standard deviation tends to fall as return period increases, there is an additional conservatism in such an approach. In countries where seasonal effects influence the road profile, at least one year will be required to get some measure of the variability due to such changes in the profile.

The uncertainty and hence the conservatism of this approach can be reduced by considering different loading scenarios separately. For example, single truck crossings and two-truck meeting events can be considered separately for both static and total load effects. The probabilities of exceedence can be subsequently combined.

It is concluded that site measurement of total load effects, including allowance for dynamics, has considerable potential as a means to assess the allowance that should be

made for dynamic amplification in bridge assessment. Assessment Dynamic Ratio (ADR) is proposed as the ratio that is necessary to convert static to total load effect for assessment purposes. This method can be used to obtain a site-specific estimate of characteristic load effect. While it is not considered here, a similar approach can be applied to estimate the distribution of lifetime maximum load effect for use in a Reliability assessment of bridge safety.

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Figures

Fig. 1. Dynamic Amplification Factor versus total strain for measured single and multiple vehicle presence (MP) events at Hrastnik Bridge.

Fig. 2. Parametric and semi-parametric fits to measured gross mass data (close ups of tail region inset) – after [14].

(a) Full CDF's of static and total bending moment.

(b) Upper tail of CDF's.

Fig. 3. CDF's of static and total bending moment for single-vehicle events.

Fig. 4. CDF's of measured static and total strain for combined database of single and multiple vehicle events.

Fig. 5. Variation in GEV fits to sets of 25 daily maxima.

Fig. 6. Extrapolation of 10 years of 5-week maxima (single-vehicle events).

Fig. 7. Variation in ADR with Return Period.

(a) Variation in ADR with Return Period.

(b) Variation in ADR with Return Period (Area Enlarged).

Fig. 8. Variation in ADR with Return Period.

Fig. 9. 5-week ADR versus 1000-year ADR for 100 different road profiles.

Fig. 10. Variation in ADR with return period (vehicle meeting events).

Fig. 11. 5-week ADR versus 1000-year ADR for 25 road profiles (two-vehicle meeting events).

Tables

Table 1. Parameters defining velocity distribution.

Mode No.	Mean (km/h)	Standard Deviation (km/h)	Relative Weight
1	76.0	14.8	0.145
2	83.0	2.6	0.487
3	88.2	1.5	0.368

Table 2. Parameters defining parametric gross mass tail fit.

Tail Region	Mean (tonnes)	Standard Deviation (tonnes)	Scale Factor
50 – 64	36	9.97	0.13
65+	24	12.35	0.99

Table 3. Correlation coefficients between ADR for various return periods and the 1000-year ADR. (These correlation coefficients are based about the line of correlation as shown in Figures 9 and 11.)

		Return Period						
		Mean	0.5 Day	Day	Week	5-Week	1-year	100-year
1000-year	Single-vehicle events	0.353	0.808	0.853	0.910	0.949	0.975	0.997
	Two-vehicle events	0.585	0.750	0.775	0.794	0.841	0.885	0.944