

Headway modelling for traffic load assessment of short to medium span bridges

Synopsis

Site-specific assessment of the loading to which existing bridges are subject has considerable potential for saving on rehabilitation and replacement costs of the bridge stock. Monte Carlo simulations, with traffic measurements from site, are used to estimate the characteristic values for load effects. In this paper, it is shown that the critical loading events from which the characteristic effects are derived, are strongly dependent on the assumptions used for the headways of successive trucks. A new approach which uses measured headway statistical distributions is developed and is shown to be a reasonable balance between conservative assumptions and less realistic scenarios. The sensitivity of characteristic load effects to conventional headway assumptions is shown to be significant.

Introduction

Methods of assessing the load carrying capacity of existing bridges are well developed in Europe and elsewhere (O'Brien *et al* 2005). However, particularly for less heavily trafficked bridges, there is often greater potential for savings in an accurate assessment of the actual traffic loading at the bridge site. In some countries such as the United Kingdom, a notional assessment traffic load model is specified. While such models can allow for some variation in traffic conditions between sites (BD21 1993, Cooper 1997), they are necessarily conservative as they are deemed to represent a wide range of situations. Bridges can often be shown to be safe for the traffic loading to which they are subject, even if they do not have the capacity to resist the notional assessment load.

Traffic loading can be determined from statistics of traffic weights, frequencies and mix and numerical modelling of truck crossing and meeting events. The statistical theories utilised in extrapolating data representing relatively short periods of time to the return period required (1000 to 3500 years), are well established (Fisher and Tippett 1928; Gumbel 1958). In recent years, these theories have been applied to the modelling of traffic loading on bridges. To aid the loading studies that were performed for Eurocode 1, Part 3, O'Connor *et al* (2001) calculated the effects induced in a bridge by the passage of traffic loads and performed a statistical analysis of these effects. Others including Bailey (1996), Grave *et al* (2000), Caprani *et al* (2002) and James (2005) have developed alternative methods of simulating loading and extrapolating the results.

There is general consensus that medium- and long-span bridges are strongly influenced by congested traffic loading, where the gaps between vehicles have become small and there is no significant dynamic interaction. For short span bridges – less than about 50m – free-flowing traffic involving a small number of vehicles with dynamic interaction, is more important. This paper considers only short- to medium-length 2-lane bridges with opposing traffic. Some authors (e.g., Nowak and Hong 1991; Nowak 1993; Ghosn and Moses 1985) have only considered 2-truck meeting events for such cases. For short bridge lengths (20 to 30m) bridges, the 2-truck meeting event is indeed important and is likely to strongly influence the bridge design. However, meetings of more than two trucks are also possible and should not be ignored. As spans get longer, the likelihood of events involving more than two trucks increases. For example, 4-truck events (with two trucks in each lane) are shown here to strongly influence the design of a 40m span bridge.

This paper describes a method for the modelling of traffic loading which allows for events involving more than two trucks simultaneously on the bridge. A key issue in this is the headway, defined as the time or distance between the front axle of a leading truck and the front axle of a following one (Thamizh-Arasan and

Koshy 2003). The term 'gap' is also used and is defined as the time or distance between the rear axle of the front truck and the front axle of the following truck. To assume too small a gap may be quite conservative whereas an excessively large gap effectively removes 3- and 4-truck meeting events from consideration.

Modelling vehicle headway

Vehicle headways are relevant to Traffic Engineering as well as bridge loading and have been the subject of research for many years (Haight 1963, Banks 2003). Several authors (Lieberman and Rathi 1992, Gazis 1974, HRB 1965) note the influence of driver behaviour on the headway distribution. The ratio of a driver's actual to desired speed and their aggression level will affect how closely they are willing to drive behind the vehicle in front. These factors also influence the likelihood of overtaking as does the flow in the adjacent lane (fewer opportunities at high flow). Vehicle performance parameters such as capability to accelerate and decelerate rapidly also play a role.

The mechanical characteristics of trucks are significantly different than lighter vehicles. In addition, truck drivers exhibit different behavioural characteristics than other drivers as a result of good route planning, commercial pressures, specialised training, high route familiarity and fatigue.

For bridge loading purposes, only trucks have significant weight and it is therefore the inter-truck headway rather than the inter-vehicle headway that is of interest. The inter-truck headway is influenced not only by truck driver behaviour but also by the number of cars between trucks. Hence, the percentage of trucks in the total traffic will result in differences between sites in the distribution of headways.

It is acknowledged that there are different headway distributions for different flows and different traffic composition (Thamizh-Arasan and Koshy 2003). At low flows, interaction between drivers takes place at greater distances (Gazis 1974). For congested traffic, Banks (2003) notes that there is considerable scatter in headway data and there is no evident relationship between headway and speeds under 100km/h.

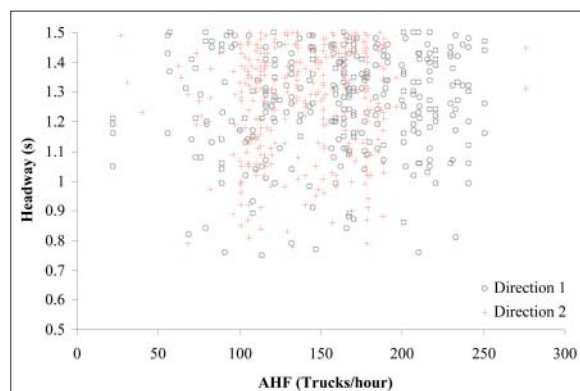
Statistical headway distributions

The Negative Exponential Distribution assumes that the arrival of vehicles is Poisson distributed (Grave 2001, Bailey and Bez 1994). Often, this distribution is shifted to the right to allow for a minimum headway and is then known as the Shifted Negative Exponential Distribution:

$$F(t) = 1 - \exp[-\gamma(t - t_0)] \quad \dots(1)$$

$$\text{where } \gamma = \frac{Q}{(Qt_0 - 1)}$$

In this expression, t_0 and Q are the minimum headway and the flow respectively. However, this formulation gives inordinately



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Fig 1. Occurrences of headways less than 1.5s and corresponding average hourly flow (AHF)

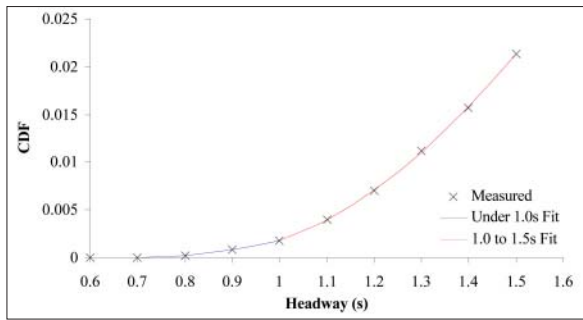


Fig 2. Cumulative distribution function (for all flows combined) and quadratic fits

high probabilities to values of headway close to the minimum allowed (Bailey 1996).

Harman and Davenport (1979) use a Uniform distribution, requiring the gap to be greater than 7.32m. Nowak and Hong (1991), Nowak (1993) and Vrouwenvelder and Waarts (1993) also assume a Uniform distribution but with different ranges. While more reasonable than the Shifted Negative Exponential, the Uniform distribution again fails to recognise the reduced probability of gaps close to the minimum.

The Gamma distribution has been used extensively in the background studies for the Eurocode for traffic loads on bridges (Bruls *et al* 1996, Flint and Jacob 1996, O'Connor *et al* 2002). However, it does not, in its left tail, take account of the driver behaviour or other factors that influence very small headways. Further, this distribution passes through the origin which is not physically possible when gap allows for the front and rear overhangs of the truck bodies beyond the axles.

Grave (2001) shows that for a given flow, the distribution of larger headways is very similar at various sites and is consistent with a negative exponential distribution. Crespo-Minguillón and Casas (1997) find similar results. They show that the distribution for 'normalised headway', defined as the headway divided by the mean headway per hour, is constant regardless of flow. The cumulative probability for headway, t , is therefore given by:

$$F(t) = \frac{Q}{3600} \left[1 - e^{-\lambda t} \right] \quad \dots(2)$$

where λ is the mean normalised headway and Q is the flow (trucks/hour). Grave adjusted the output by changing all gaps less than a specified minimum to the minimum value. The Normalised Headway approach is good for larger headways but is an inaccurate representation of the distribution for smaller headways.

Representing headway distribution statistics

For this project, statistical distributions are fitted to the measured headways (Headway Distribution Statistics – HeDS) to accurately represent actual site conditions. Five days of Weigh-In-Motion (WIM) data was processed from the two outermost (slow) lanes of the 4-lane A6 motorway near Auxerre in France. The average hourly truck flows were 3336 and 3408 for directions 1 and 2 respectively.

The flow and hence the headway distribution varies considerably by hour. For each hour of each day and each direction, headway data and the corresponding hourly flow (mean flow in an hour) are noted. Data from different hours are combined according to intervals of hourly flow and cumulative distribution functions calculated for each interval.

For headways of less than 1.5s, the correlation between hourly flow and headway is weak as can be seen from Fig 1. Hence, it is reasonable to assume a distribution of headway that is independent of flow. This approach is supported by the theory that, for small headways, driver perception of safe distance rather than traffic flow determines the headways (Koppa 1992; Lieberman and Rathi 1992). Combining all available headway data less than 1.5s, for both directions, gives the cumulative distribution function illustrated in Fig 2. This measured distribution is fitted with two quadratic equations, one for less than 1s and another between 1 and 1.5s.

For headways between 1.5s and 4s, there is a correlation between headway and flow. All available data are categorised by hourly flow in intervals of 10 trucks/h. The resulting cumulative

distribution functions (cdf's) for headway are illustrated in Fig 3. There is a general trend of increasing cumulative probability – for all headways in this range – with increasing flow. There is some variation about this trend which seems likely to be a result of the relatively small data set.

The truck flows change by hour and by day as can be seen in Fig 4. To simulate a 'typical' day, the average hourly flow (AHF) from the five days of traffic data is calculated for each hour and direction (black curve). In Fig 3, a quadratic equation is fit to the measured Cumulative Distribution Function closest to the AHF for each hour. By fitting to the measured headway distribution of Fig 3 for each AHF, there is faithful adherence to the measured data. No attempt is made to identify and fit to the relationship between cumulative distribution and flow for a given headway.

There is a noticeable change in the slope of the cdf's at 4s. A truck travelling at a typical highway speed of 80km/h travels 89m in 4s. Hence for bridges up to 50m long, headways in excess of four seconds are generally not critical and do not need to be modelled as accurately as smaller headways. Therefore, the normalised headway distribution of Grave (2001) was used.

Impact of headway model on critical loading events

A range of short- to medium-length bridges are considered; 20, 30, 40 and 50m. In each case, three load effects are considered:

- Load Effect 1: Bending moment at the mid-span of a simply supported bridge;
- Load Effect 2: Left support shear in a simply-supported bridge;
- Load Effect 3: Bending moment at central support of a two-span continuous bridge.

50 days of traffic are simulated and the maximum daily load effects identified in each case. The results obtained using the fits to HeDS described above are taken as the reference for assessing the validity of other assumptions. This process was carried out five times to provide an indication of the repeatability of results. Five sets of 50 days are also simulated for four minimum gap assumptions: 5m, 10m, 0.5 s and 1.0 s. In each of these cases, Eqn 2 is used, adjusted for gaps less than the minimum, as described by Grave (2001). The errors in the mean of the 50 daily maxima are illustrated in Fig 5 for Load Effect 1. Five points are shown in each case, indicating the random variation between each run of 50 days. It can be seen that the daily maxima are quite sensitive to the gap assumption, particularly for the longer spans. For the load effect illustrated, most gap assumptions are conservative; up to about 15% in the case of the 5 m assumption. Errors for Load Effect 3 are similar to Load Effect 1 while those

Fig 3. Cumulative distribution functions between 1.5s and 4s for range of hourly flows

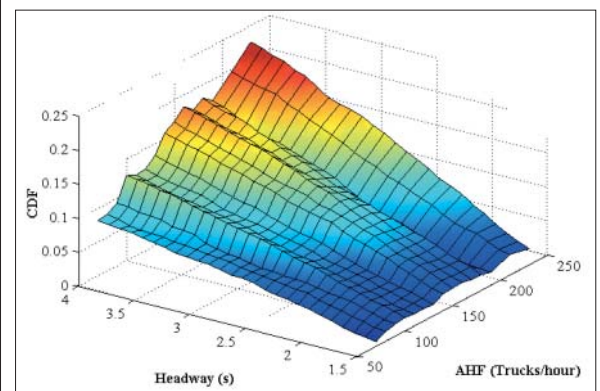
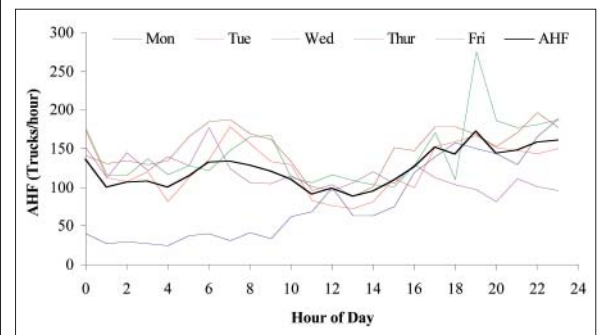


Fig 4. Daily variation in average hourly truck flow for Direction 2 (AHF = Average Hourly Flow)



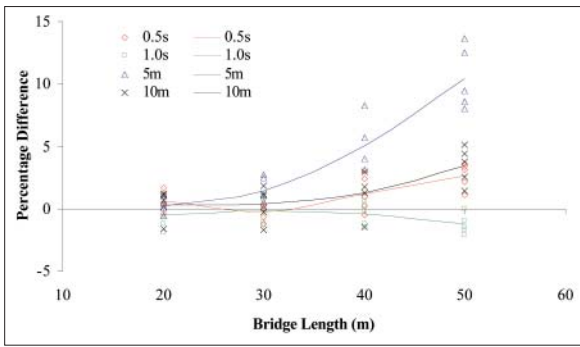


Fig 5. Percentage difference in mean of daily maxima for Load Effect 1 (mid-span simply-supported bending moment) relative to HeDS

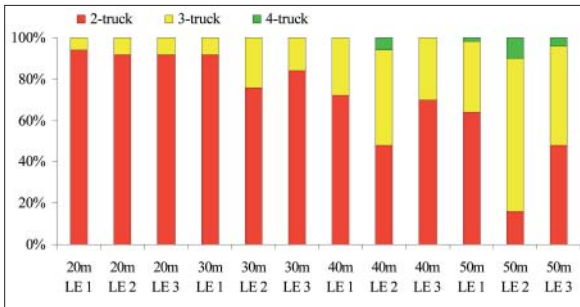


Fig 6. Number of trucks involved in maximum-per-day load effects (LE i = Load Effect i) (HeDS, Run 1)

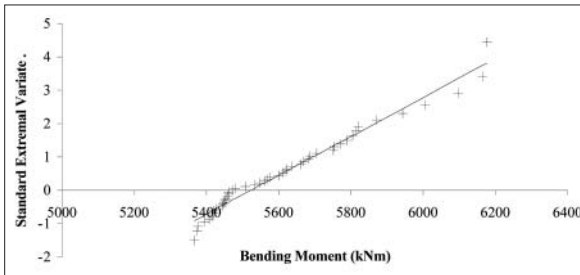


Fig 7. (left) Plot of daily maxima for HeDS, Run 1, Load Effect 1, 30m bridge, on probability paper (straight line indicates compliance with Extreme Value Type 1 distribution)

for Load Effect 2 are more variable and can be conservative or non-conservative.

Statistics for the numbers of trucks involved in the daily maximum load effects are shown in Fig 6. There are no single truck loading events that feature in the daily maxima but 2-truck events are important, particularly for shorter lengths. 3-truck events are also present in the population of daily maxima, particularly for greater lengths. This is most pronounced for Load Effect 2 (mid-span moment in a 2-span bridge) where the influence line favours a lesser concentration of loading. Surprisingly and contrary to assumptions made in some past studies, 4-truck events feature significantly for 40m and 50m bridges. Similar graphs for the other gap assumptions (Caprani 2005) show that these also feature significant frequencies of 4-truck and even 5-truck events which is consistent with the finding (Fig 5) that they are conservative.

It is not the mean of the daily maximum load effects that is used in bridge assessment but the load effect with an acceptably low probability of exceedance, i.e., the characteristic value. The daily maxima for a typical load effect are plotted on probability paper for the Extreme Value Type 1 distribution (Gumbel 1958) in Fig 7. Assuming 250 working days/year, the daily maximum with a probability of exceedance of 1/250 000 is the characteristic value for a 1000 year return period (as used for Eurocode 1). Setting the probability to this level and inverting the cdf gives the characteristic load effect. This process corresponds to fitting a straight line to the points in Fig 7 and extending it until it intersects with a standard extremal variate (y-axis) value of 12.43. Other statistical distributions may fit better to the data. However, the Type 1 assumption is used here to provide a comparative measure which takes account not only of the average magnitude of the daily maxima but also of the slope of the line.

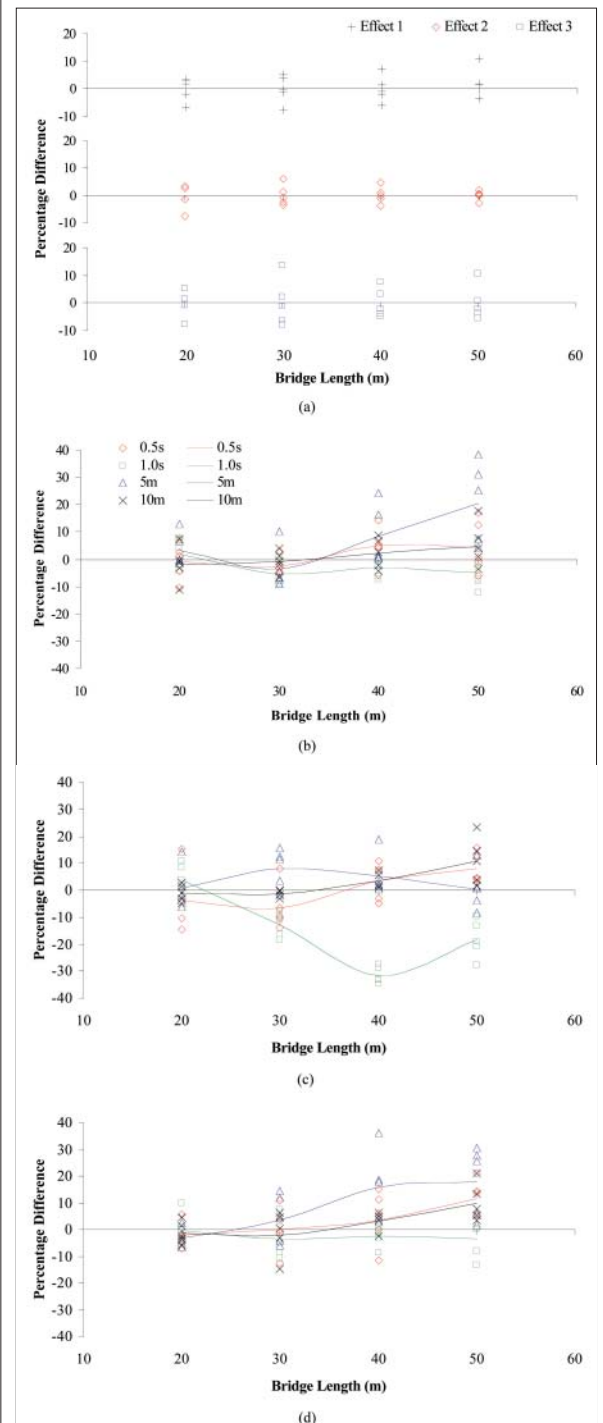
The process is quite sensitive to the truck weight data used and would generally require considerably more than the 5 days of weight data used here. Nevertheless, 1000-year characteristic values are calculated to provide comparative results using the same weight histogram in each case and various gap assumptions.

Fig 8. (right) Variation in characteristic values calculated using different gap assumptions (a) Variation in HeDS results (b) Load Effect 1 relative to mean HeDS value (c) Load Effect 2 relative to mean HeDS value (d) Load Effect 3 relative to mean HeDS value

Characteristic values are calculated from 50 days of simulations using measured headway distributions (HeDS) and for each of the gap assumptions. All calculations are carried out five times to provide a measure of repeatability. The five HeDS results for each bridge length appear as points whose mean is on the x-axis in Fig 8 (a). The results relative to the mean of the five HeDS calculations are presented in Figs 8 (b) to (d). For the different gap assumptions, there is significant variation in the results for each of the five runs, particularly for longer bridges. In general, there is considerably less variation in the five HeDS runs.

The gap assumptions result in substantial differences in characteristic value. While there is considerable variation between runs, it can be seen that the gap assumption strongly influences the mean result – by as much as 20% to 30%, depending on the assumption adopted. For Load Effects 1 and 3, the traditional approaches are generally conservative but for Load Effect 2 there is no clear trend.

There is a substantial non-conservative result from the 1.0s gap assumption in 40m bridges. The critical distance between



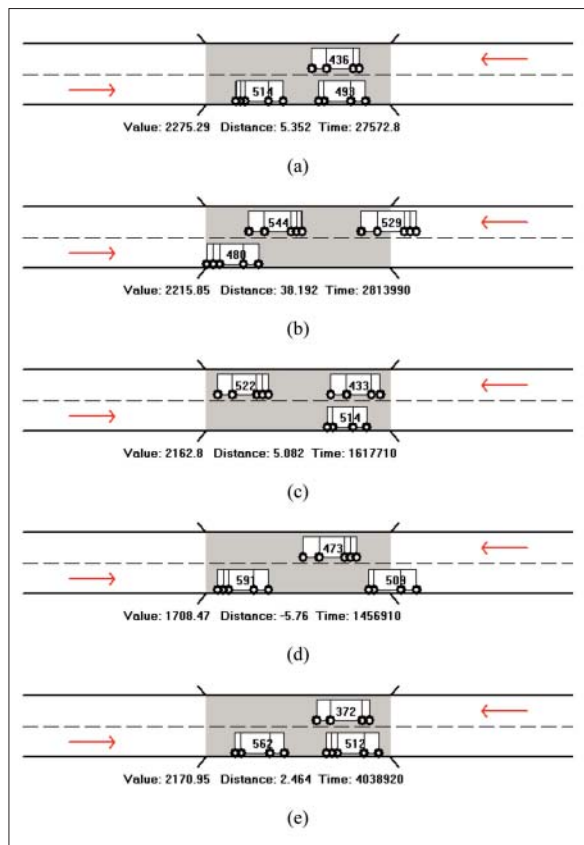


Fig 9. The critical 3-truck daily maxima for Load Effect 2 and 40m length for gap assumptions (a) 5m; (b) 10m; (c) 0.5s; (d) 1.0s; and (e) HeDS. GVW is shown on the trucks in units of deci-tonnes (i.e. kN/0.981)

point loads for central support moment in a bridge with two 20m spans, is a little less than 20m. The truck speeds are Normally distributed but generally within 70 to 80km/h, equivalent to 19.4 to 22m in 1s. Given a 10m wheelbase for the following truck, the minimum distance between the two tridem implied by the 1.0s minimum gap is 29 – 32m. As a result, load effect 2 will be underestimated for spans in the range 35 – 45m. Fig 9 illustrates this point; it can be seen that the 1.0s gap assumption has resulted in a considerably different truck arrangement to other methods.

Conclusions

Vehicle headways are identified as a key input into the statistical modelling of traffic loading on bridges. Direct fitting to measured headway distributions is developed as an accurate method of addressing this issue. Headways of less than 1.5s are found to be insensitive to flow and fit well to quadratically-increasing cdf curves. In the 1.5 to 4.0s range, headway probability is found to be strongly influenced by flow. The average flow in each hour of the day is calculated and the corresponding cumulative distribution function used. A negative exponential distribution is used for greater headways, as described by other authors. The procedure is demonstrated using measured truck weight and headway statistics from a highway in France.

Many assumptions on the statistical distribution of gaps and headways have been adopted in the past. It is shown that the gap assumption makes a significant difference to the key maximum-per-day load effect data. Characteristic values are even more sensitive to the assumption adopted and the errors can be conservative or non-conservative depending on the span, load effect and assumption adopted.

The authors recommend a HeDS approach for site-specific assessment of bridge loading. Sufficient weight data should be used to allow for the high sensitivity of characteristic load effects to GVW data and seasonal trends where these exist. Other headway assumptions may be adopted but, given the greater variation in results, the calculations need to be repeated a number of times to quantify and allow for this uncertainty.

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