

Critical Loading Events for the Assessment of Medium Span Bridges

C.C. Caprani*, S.A. Grave†, E.J. OBrien+, A.J. O'Connor‡

*Post-graduate Researcher, University College Dublin, Ireland,

†Post-graduate Researcher, Trinity College Dublin, Ireland,

+Professor of Civil Engineering, University College Dublin, Ireland,

‡ Lecturer, Trinity College Dublin, Ireland.

Abstract

This paper describes the simulation of free-flowing traffic across bridges to predict the characteristic values for bridge load effects such as bending moment and shear force. The results of these simulations are then used to demonstrate that, in predicting the characteristic extreme load effects to which a bridge may be subjected, it is not sufficient to solely model one- or two-truck presence events. It is shown that loading events involving three or more trucks may need be included in the model for short to medium spans. The critical loading events for a particular load effect are strongly dependent on the span and the shape of the influence line.

Keywords: Bridges, Loading, Traffic, Trucks, Statistics, Simulation.

1 Introduction

Considerable attention has centred in recent years on the assessment of the load carrying capacity of existing bridges. In many cases, it is possible to assess carrying capacity reasonably accurately. However, the appropriate level of applied traffic loading on bridges for a given site can be considerably more difficult to determine. The traffic loadings which new bridges are required to carry are generally notional and, consequently, can be excessively conservative. In such cases, great savings can be achieved for existing bridges through the accurate assessment of the applied traffic load. O'Brien & Jacob [1] report that Weigh-In-Motion (WIM) technology allows the parameters governing traffic flow to be accurately measured and thus realistic simulations can be performed in which multiple truck presence events (MTPE's) are modelled. This will result in more realistic estimates of the governing load effects. However it is accepted that WIM sensors still do not give fully accurate estimates of weights [2].

The recent developments in the assessment of bridge loads has led to more rational statistical approaches in bridge design codes. In the late 1980's the studies for the draft Eurocode 1: Part 3, *Traffic loads on bridges* (EC 1.3) [3], began and

this interest, coupled with advances in the accuracy of WIM technology, has greatly improved the accuracy of calculated traffic loads to which bridges may be subject. To aid the original studies of EC 1.3, Eymard & Jacob [4] developed simulation software called CASTOR-LCPC (Calcul des Actions et Sollicitations du Trafic dans les Ouvrages Routiers - Laboratoire Central des Ponts et Chaussées). The function of CASTOR is the calculation of effects induced in a bridge by the passage of traffic loads on the deck and the statistical analysis of these effects through the use of histograms. More recently Bailey [5] and Grave [6] have developed similar applications, extended into their respective areas of interest.

The statistical theories utilised in extrapolating the results of relatively short periods of simulation time (normally around two weeks) to the return period required have been known for many years [7, 8]. However it was not until recently that the use of these theories, the circumstance in which they may be used and the accuracy that they may provide was examined with the specifics of modelling bridge traffic loading in mind [9].

It is generally assumed, that the one- and two-truck free-flowing events are the most important events for short to medium span two-lane bridges. This assumption is tested here. It is shown that, up to 50 m span, two or three trucks present on the bridge simultaneously are generally the critical load case. Further, it is shown that the critical truck load arrangement depends on the characteristics of the influence line considered. For influence lines with less pronounced peaks, the three-truck event tends to be important whilst for other influence lines it may be sufficient to consider the two-truck event only. This paper describes the critical truck loading events involving one-, two- or three-trucks, which are critical for different ranges of span.

2 Simulation of bridge loading events

2.1 Generating Traffic Files

For this study real-time vehicle weights and frequencies were measured at a site in France using WIM technology. The data was analysed to determine the parameters of the statistical distributions that characterise that traffic flow [10]. The characteristics measured include the Gross-Vehicle-Weight (GVW), speed, headway, number of axles, flow rates, inter-axle spacing and the weight of each axle. All subsequent calculations based on this data are inherently site-specific; however, the method adopted is general and thus is applicable to any site whose traffic characteristics are known.

The site chosen was the RN10 near Angers, France. The WIM data was recorded for one week from 7th to 14th April 1987. The site has 4 lanes of traffic (2 in each direction) but traffic was recorded in the slow lanes only. The simulations performed in this work represent two opposing lanes of traffic on a two-lane bridge.

The authors have developed an object-orientated program, written in the C++ language. Monte-Carlo simulation is used to generate a traffic file whose statistical distributions closely match those of the measured data. The closeness of the match

depends on the number of trucks generated which in turn depends on the vehicle flow rates for the site and the duration for which the traffic is being simulated.

The characteristic value of a load effect required in the design or assessment of a bridge is that with a probability of occurrence of once in a designated period - the return period. The return period is generally quite a large period of time. For example, EC 1.3 recommends a period of 1000 years for bridges or monumental structures. In this study the period of traffic simulated was two weeks and the resulting number of trucks in each traffic file was approximately 32,100.

2.2 Simulating Crossing Events

A simulation was carried out of the traffic flow over the bridge length under consideration. In order to minimise the processing requirements, MTPE's and single trucks with GVW in excess of 50 tonnes were identified and only in these situations were further calculations performed. The complete traffic file was examined, vehicle by vehicle, to identify all such cases.

The simulations use influence lines to calculate the value of the load effects for any position and arrangement of truck(s). In this study two load effects were considered (Figure 1):

- Effect 1: Bending moment at the mid-span of a simply supported bridge.
- Effect 2: Bending moment at the central support of a two-span continuous bridge.

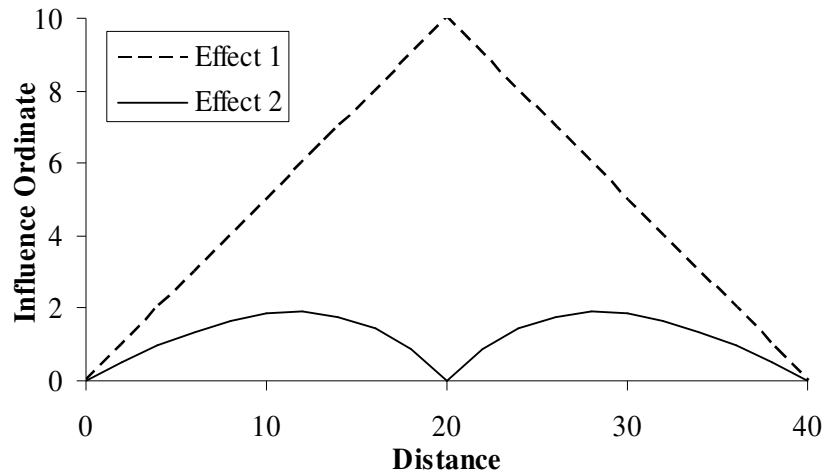


Figure 1: Influence lines for a 40 m bridge length: bending moment at centre of simply supported (Effect 1) and two-span continuous (Effect 2) bridge

These two load effects were considered due to the differing nature of their respective influence lines. For each MTPE and single truck with GVW over 50 tonnes, the trucks were moved in 0.01 second intervals across the bridge and the maximum load effects for the event identified. For this study, the errors that result from neglecting three-truck events are shown. This was done by considering two cases:

- EV12: Only single- and two-truck events are considered,
- EV123: One-, two- and three-truck events are considered.

In both cases, the results were processed to determine the characteristic 1000-year load effects.

3 Prediction of extreme load effects

The load effect caused by the passage of trucks is a random variable and the results of the calculations of all the events noted above gives a parent population of an undetermined statistical distribution. The extrapolation from the two-week sampling period to the 1000 year return period is achieved using extreme value statistics. It was decided to use the maximum hourly load effect as the basis for the extreme value distribution population. These hourly maxima are themselves random variables conforming to an extreme value distribution [11-13].

Traffic was simulated for 24 hours per day for ten working days, deemed to represent two weeks. Hence, 240 maximum values for each load effect were calculated. However, as no events occurred in some hours, a number of the hourly maxima had a value of zero. The maximum load effects per hour were assumed to comply with the Extreme Value Type I (Gumbel) distribution. The cumulative distribution function (CDF) for the Gumbel distribution is given by:

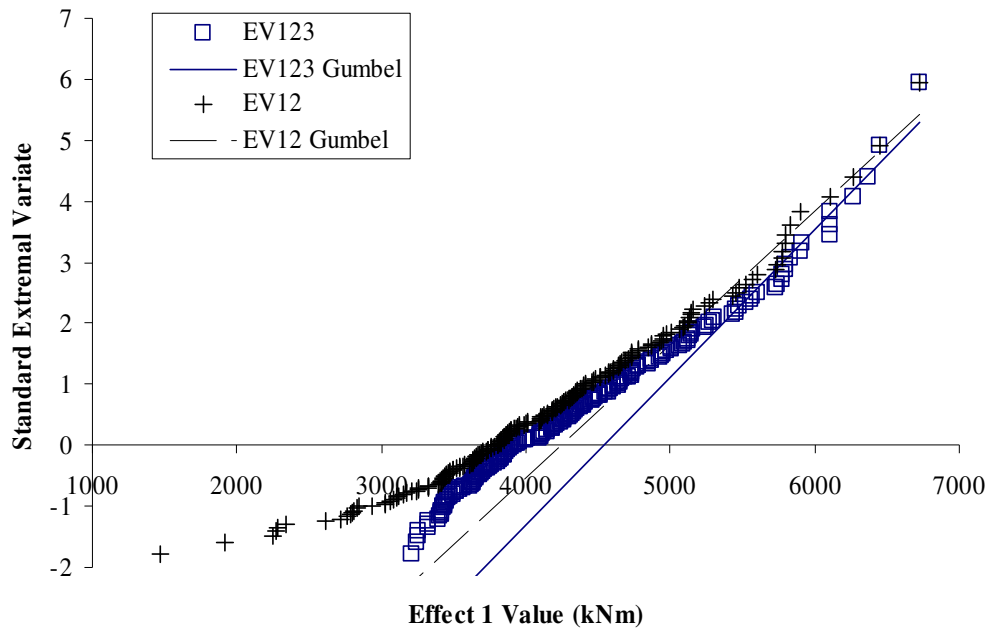
$$F(x) = \exp\left[-\exp\left(\frac{-(x-\lambda)}{\delta}\right)\right] \quad (1)$$

where λ and δ are the parameters that characterise the distribution. Probability paper was used to test the conformity of the hourly maxima with this distribution. This process consists of ranking the maxima in ascending order and calculating a plotting position for each point. Goda [14] has suggested using the Gringorten plotting position for engineering applications, given by:

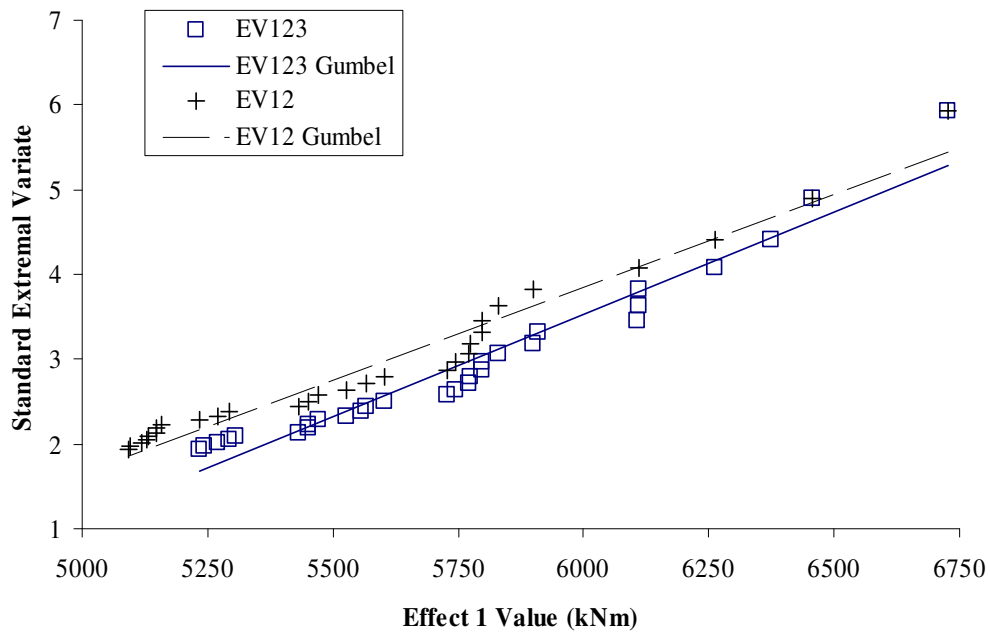
$$p = \frac{i - 0.44}{n + 0.12} \quad (2)$$

where i is the rank ($i = 1$ is the lowest value of the maxima) and n is the number of non-zero data points. The y ordinate is then given by:

$$y = -\ln[-\ln(p)] \quad (3)$$



(a) All non-zero points



(b) Tail data used in least squares fit

Figure 2: Gumbel plot for Effect 1 with a bridge length of 40 m

The x ordinate in the graph is simply the value of the variable. A linear trend indicates good compliance with the assumed distribution. Castillo [12] suggests that only k of the tail values should be utilised for extrapolation purposes and suggests a value of:

$$k = 2\sqrt{n} \quad (4)$$

Figure 2(a) illustrates the Gumbel plots for EV12 and EV123. In this case the number of non-zero data points is 211 and thus k is 29. Figure 2(b) illustrates the k tail points and resulting least-squares-fit line. In circumstances where the tail values are not linear, other extreme value distributions are available. The linearity of the plot in the tail region is evidence of a reasonable approximation to a Gumbel distribution. The slope and intercept of the line correspond to the parameters of the distribution as follows:

$$m = \frac{1}{\delta}; \quad c = -\frac{\lambda}{\delta} \quad (5)$$

where m is the slope of the line and c is the intercept. Having determined these parameters, the value of a load effect can easily be calculated for a specified return period. Allowing for 250 working days per year, there are 6×10^6 hourly maxima for a 1000-year return period.

4 Simulation Results

To assess the repeatability of the procedure, six full simulations were carried out using the procedures outlined above. The two-week traffic files were run for Effects 1 and 2 and for bridge lengths of 20, 30, 40 and 50 m. The results of these simulations for EV12 and EV123 events are given in Table 1. The variability of the characteristic values of the six different runs is illustrated in Figure 3 for EV123. It can be seen that the results are reasonably consistent between runs.

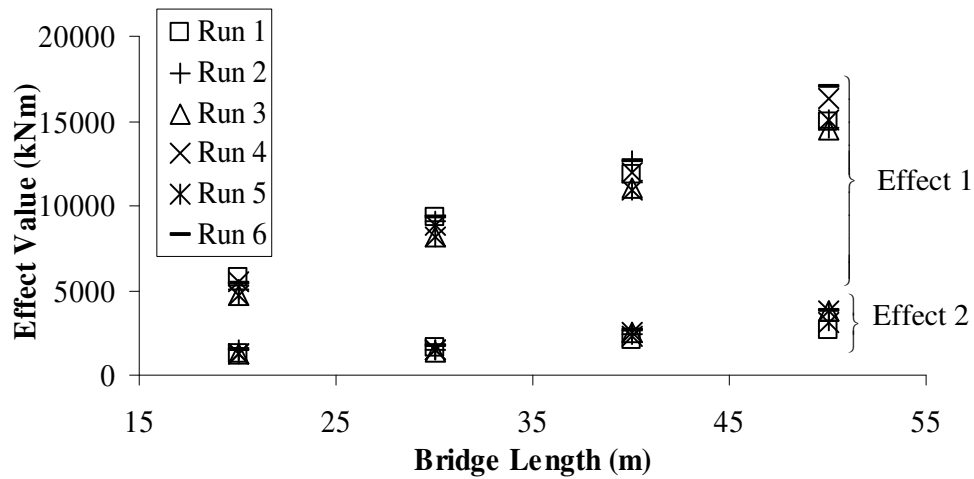


Figure 3: 1000-year extrapolated values for EV123

Length	Effect	Event	Simulation No.					
			1	2	3	4	5	6
20 m	1	EV12	6002	5186	4852	5616	4789	5384
		EV123	5798	5108	4770	5546	4685	5390
	2	EV12	1354	1417	1310	1294	1265	1507
		EV123	1292	1440	1293	1280	1241	1489
30 m	1	EV12	9692	9348	8255	9262	8082	9397
		EV123	9286	9125	8152	8864	8215	9258
	2	EV12	1653	1566	1352	1419	1436	1852
		EV123	1590	1467	1396	1474	1550	1768
40 m	1	EV12	12622	12627	11376	13032	10688	13281
		EV123	11822	12644	10991	11933	10935	12691
	2	EV12	2170	2266	1950	1927	2275	2184
		EV123	2033	2385	2562	2251	2567	2675
50 m	1	EV12	16337	15831	14339	15778	13861	17108
		EV123	14933	14596	14529	16374	15040	16969
	2	EV12	2581	2937	2554	2839	2870	2670
		EV123	2599	3226	3791	3139	3766	3812

Table 1: 1000-year extrapolated values for all simulations

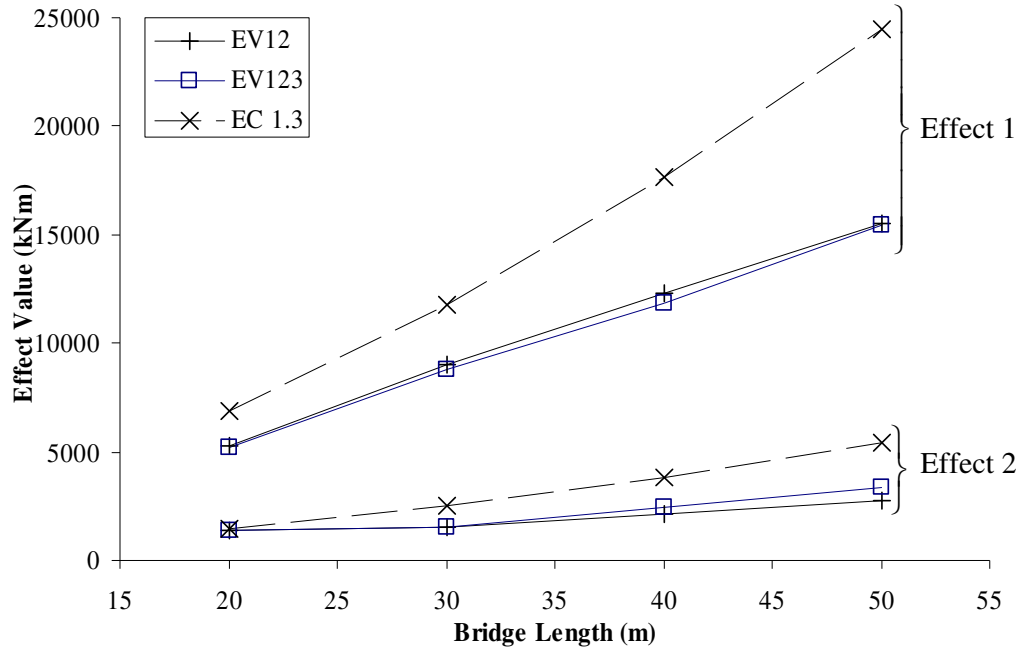


Figure 4: 1000-year extrapolated values plotted against length

The mean values for all six runs are given in Table 2 and Figure 4. For comparative purposes, the corresponding design values in accordance with the Normal load model of EC 1.3, are also illustrated in the figure. For these calculations, a prismatic

section and a carriageway width of 7.5 m was assumed. As would be expected, the Eurocode values are substantially greater than the site-specific characteristic values. The relative differences are also presented in Table 2.

Length	Effect	Event	Mean	EC 1.3	Difference (%)
20 m	1	EV12	5305	6913	30
		EV123	5216	6913	33
	2	EV12	1358	1440	6
		EV123	1339	1440	8
30 m	1	EV12	9006	11803	31
		EV123	8817	11803	34
	2	EV12	1546	2519	63
		EV123	1541	2519	64
40 m	1	EV12	12271	17650	44
		EV123	11836	17650	49
	2	EV12	2129	3837	80
		EV123	2412	3837	59
50 m	1	EV12	15542	24443	57
		EV123	15407	24443	59
	2	EV12	2742	5394	97
		EV123	3389	5394	59

Table 2: 1000-year extrapolated values and Eurocode results

The differences which arise from neglecting three-truck events can also be seen in Figure 4. For Effect 2, particularly for longer bridge lengths, there is a reduction in the characteristic value when the three-truck events are ignored. This is to be expected as a number of extreme loading situations have been omitted from consideration. The difference between the two load cases increases to 19.1% for a bridge length of 50 m. For Effect 1, however, neglecting three-truck events tends to result in an increase in the characteristic values. This difference is quite small; the maximum difference is 3.7% at a length of 40 m. The increase results from a crossing of the lines which best fit the distribution tails (see Figure 2).

Figure 5 illustrates the six most critical events for Effect 2 and a bridge length of 40 m. These events are taken from the results of Simulation No. 1. The most critical event involves two trucks travelling in the same direction (one in the first span and one in the second). As there are only two trucks, this features as the most critical event in both EV12 and EV123. The next three most critical events involve three trucks. Hence, the second most critical event in EV12 is the fifth most critical in EV123. It is clear from this that, for a bridge length of 40 m and for Effect 2, consideration of three events is important to determine the characteristic load accurately in an assessment of the structure. It may also be noted that single truck events do not feature in the top six events for this load effect. For other influence lines this may not be the case and as such it is considered prudent to ensure that one

truck events are modelled in any assessment of the characteristic load for any effect under consideration.

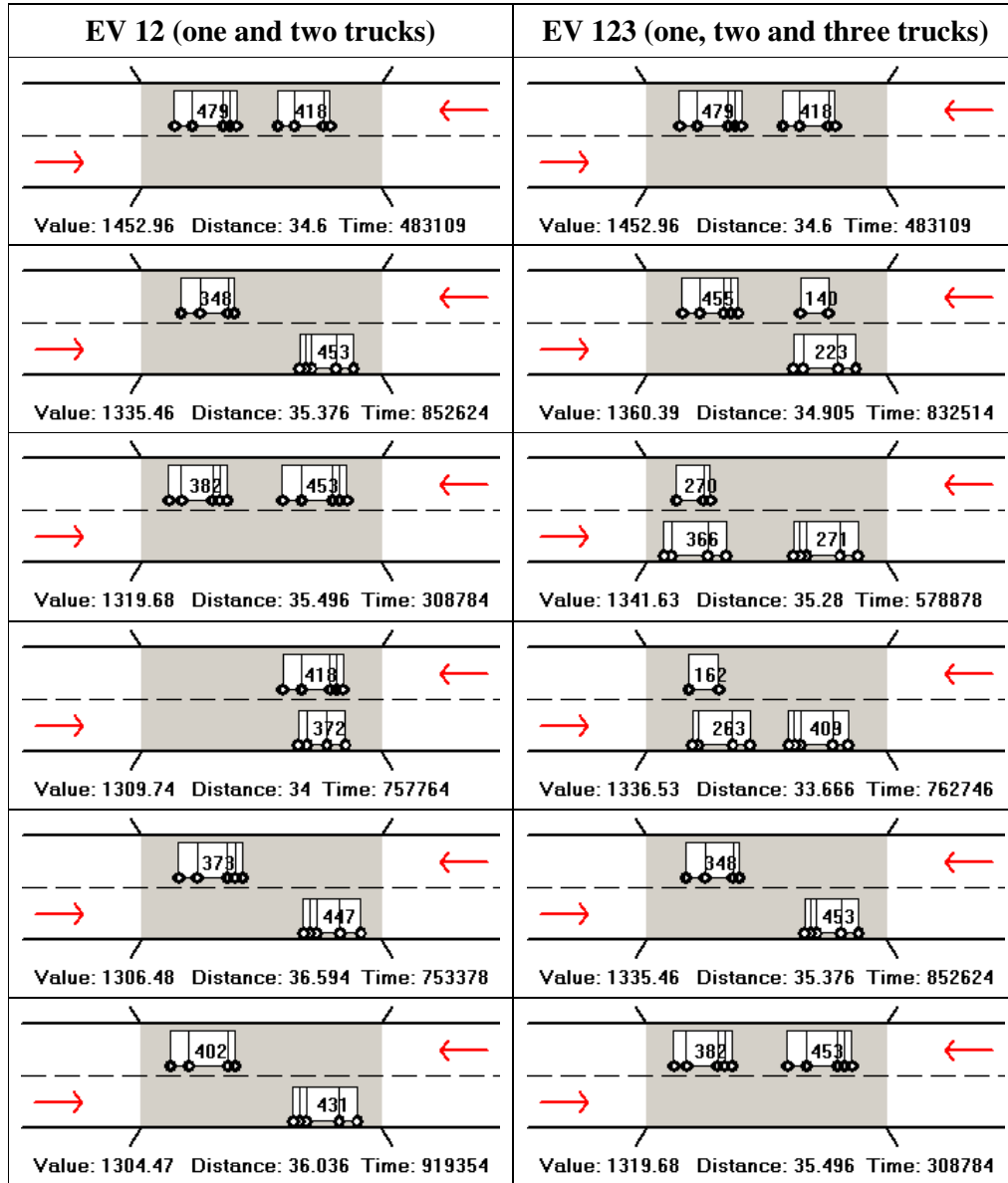


Figure 5: The six most critical events for Effect 2 and a bridge length of 40 m

It is clear in Figure 4 that the implications of neglecting three-truck events depends quite significantly on the effect considered. As the same traffic was used, this can only be attributed to the characteristics of the influence line under consideration. Figure 6 shows that Effect 2 has a less pronounced peak in the influence line than Effect 1. It can also be seen that the length for which the influence line ordinate exceeds 80% is 20% and 44% of the bridge length for Effects 1 and 2 respectively.

Thus for Effect 2 there is a greater chance that three trucks may be located in the critical zone to induce a larger effect than two trucks. This fact may account for the differences between Effect 1 and 2 in Figure 4.

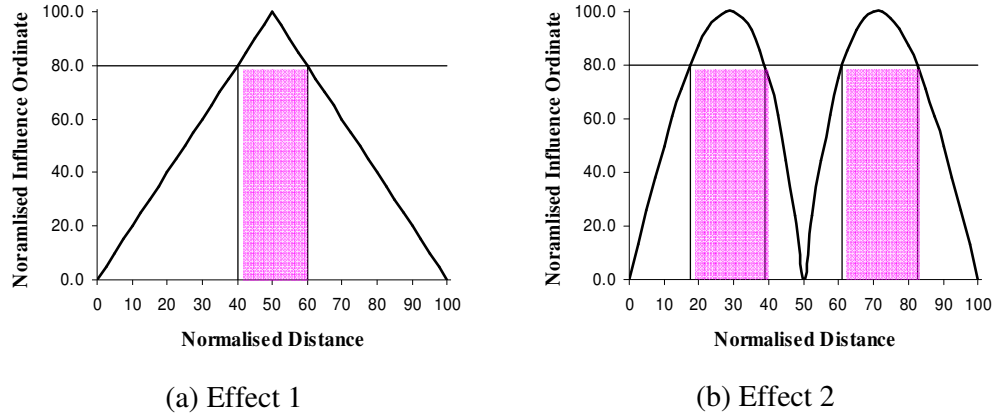


Figure 6: Attributable distances for influence ordinates $\geq 80\%$ of maximum value

It is of note that traffic jam situations have not been examined in this study and, in order for specific recommendations to be made regarding characteristic load effects for this site, the jammed loading situation would have to have been examined. It has previously been demonstrated that the bridge length at which jammed conditions become critical is site specific [9, 15].

5 Conclusions

This paper identifies the load cases that govern the assessment of traffic loading for short to medium span bridges. It is shown that in bridges of two-opposing lanes, the two-truck event is the most important free-flowing event. However, events involving three or more free-flowing trucks can be significant. This is particularly so for longer spans and for load effects where the influence line is not peaked. In such cases congested conditions may also start to feature. It is shown that in certain circumstances the inclusion of the third truck may actually decrease the extrapolated characteristic load. Thus the authors conclude that in assessing site-specific bridge loading for bridge lengths up to 50 m and in free flowing situations, both two and three truck events should be modelled.

6 Acknowledgements

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