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A Pseudo-Microsimulation Approach for Modelling Congested Traffic Loading on Long-Span Bridges

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ABSTRACT

This paper investigates congested traffic loading on long-span bridges through the use of traffic microsimulation. Six months of Weigh-In-Motion (WIM) free-flow traffic data (including cars) are used as input for the microsimulation of congested traffic. Key parameters that affect traffic loading are identified in the output of the microsimulation, and these parameters form the basis for a more computationally-efficient ‘pseudo-microsimulation of congested traffic’ (PMCT) model. This PMCT model is shown to replicate the traffic loading from full microsimulation accurately and allows long-run simulations, equivalent to 1000 years of congested traffic, to be performed with an acceptably short duration. This reduces the significant uncertainties associated with extrapolating short-run simulation results to long return periods. The 1000-year simulated results from the PMCT are compared with the extrapolated results from full microsimulation, and with the traffic loading from some design codes, for different bridge lengths. Both types of microsimulation are also applied to calculate maximum lifetime loading for two typical long-span bridges – one cable-stayed and one suspension bridge – using influence lines determined from finite element models.

Keywords: Bridge loads; Bridges, Long-span; Traffic engineering; Probabilistic models; Simulation models.

1 Introduction

1.1 Background

Traffic volumes and vehicle characteristics are expected to alter over the coming years with increasing numbers of freight trucks travelling on European Highways (European Commission 2009). The annual cost of maintaining the European bridge stock is estimated to be €6.6 billion (COST 345 2004) and with increasing traffic volumes maintenance costs will increase (Mattsson et al. 2007). Long-span bridges are particularly expensive to maintain due both to their size, which makes repair more costly, and their strategic nature, which increases the economic cost associated with the disruption of traffic flow while the bridge is being rehabilitated. In any highway bridge structure assessment, traffic loading is one of the most variable parameters and, accordingly, its accurate assessment can have a major impact on the identified rehabilitation needs of the structure. The Eurocode Load Model 1 (EC LM1) was originally calibrated for use on bridges up to 200 m long and is described in the Eurocode as being conservative for longer bridges (EC1.2 2003). Given the level of expenditure involved, a more accurate traffic load model is required for long-span bridges to avoid the cost associated with overly conservative bridge rehabilitation programmes, and the premature replacement of significant bridges.

1.2 Long-span bridge traffic load models

For long-span bridges, the governing form of traffic loading is often assumed to be congested traffic (Buckland et al. 1978, Buckland 1981). It is considered static in nature, or to be travelling at very low speed (OBrien et al. 2015a, Caprani et al. 2016), such that no allowance is needed for dynamic load effects. Previous modelling of congested traffic has tended to be based on some simplifying and conservative assumptions with regards to vehicle characteristics, vehicle arrival processes, inter-vehicle gaps, vehicle speeds as they traverse a bridge, and extrapolation of load effects to estimate lifetime maxima.

Early methods of recording vehicle characteristics such as weight and length involved stopping vehicles and recording their data at static weigh stations (Buckland et al. 1978, Buckland 1981, Ivy et al. 1953). Traffic parameters such as flow and arrival of successive vehicles were obtained through the use of video recordings (Buckland et al. 1978) or surveys (Flint and Neill 1986). The use of early Weigh-In-Motion (WIM) systems was expensive and therefore long recordings of traffic were not easily obtainable, with recording times for some models as low as 10-days (Flint and Neill 1986). However recent advances in WIM technology have made the recording of data more accurate and less costly. This allows for longer recording times such as Enright et al. (2013), which utilized traffic data from 11 WIM stations with over one year of data from each station.

Different approaches have been adapted to model vehicle characteristics important to bridge loading, particularly for trucks which, with their higher Gross Vehicle Weights (GVWs), dominate traffic loading. Although cars are relatively light, their presence does have a significant bearing on the spatial layout of trucks. Truck weights and lengths have either been taken directly from recorded traffic (Ivy et al. 1953, Harman et al. 1984, Nowak et al. 2010), or from statistical distributions of truck lengths and weights (Flint and Neill 1986, Crespo-Minguillon and Casas 1997, OBrien et al. 2010). Light goods vehicles (LGVs) are represented in different models by:

- A uniformly distributed load (UDL) of 2.0 kN/m (Harman et al. 1984)
- A fixed length and weight of 5.5 m and 3.5 t (Flint and Neill 1986)
- A fixed length and weight of 6 m and 4 t (OBrien et al. 2010)
- Taking the LGVs weights and lengths directly from the recorded traffic (Nowak et al. 2010)
- Creating a set of standard LGVs (Croche and Salvatore 1998, 2001)
- Using a statistical distribution of LGV lengths and weights (Crespo-Minguillon and Casas 1997).

Cars are disregarded completely in some models (Vrouwenvelder and Waarts 1993, Croche and Salvatore 1998, 2001), whereas others represent them by:

- A UDL of 2.0 kN/m (Harman et al. 1984),
- A fixed length and weight of:
 - 4.9 m and 1.59 t (Buckland et al. 1978, Buckland 1981),
 - 4.5 m and 1.2 t (Flint and Neill 1986)
 - 4.3 m and 0 t (i.e. for spacing purposes only) (Vrouwenvelder and Waarts 1993)
 - 4.5 m and 1.5 t (OBrien et al. 2010)
- a deterministic car length and a mean car weight (Ditlevsen and Madsen 1997)
- Using a statistical distribution of car lengths and weights (Crespo-Minguillon and Casas 1997).

The arrival of successive vehicles has generally been taken to be independent and random. This is the case in the background studies to the Eurocode (Bruls et al. 1996, Flint and Jacob 1996). Ivy et al. (1953) and Nowak et al. (2010) maintained actual vehicle patterns as recorded. Vrouwenvelder and Waarts (1993) modelled trucks statistically from traffic recordings and Crespo-Minguillon and Casas (1997) used a Markov vehicle arrival process based on a transition matrix computed from traffic measurements. In recent research, traffic microsimulation has been used to simulate traffic breakdown from free-flow to congested, and subsequently obtain congested traffic loading (OBrien et al. 2010, Chen and Wu 2011, Caprani 2012, Enright et al. 2013, OBrien et al. 2015a).

For some models a fixed inter-vehicle gap (back axle of front vehicle to front axle of following vehicle) is used; for example, Nowak et al. (2010) use a gap of 7.6 m between trucks. Nowak and Hong (1991) modelled assumed gaps of both 15 ft (4.57 m) and 30 ft (9.14 m). Vrouwenvelder and Waarts (1993) used two gap models: a gap of 5.5 m for distributed lane loads and a variable gap of 4 to 10 m for full modelling. In the background studies for the Eurocode a fixed gap of 5 m was used (Bruls et al. 1996, Flint and Jacob 1996, Prat 2001). Flint and Neill (1986) used gaps of 0.9 m for groups of 10 or fewer vehicles, increasing linearly to 1.8 m for groups of 100 vehicles. Bailey (1996) used a beta distribution to model gaps in congestion, with the mode of the distribution giving a bumper-to-bumper gap of approximately 6.4 m, with a minimum of 1.2 m. Harman and Davenport

(1984) randomly selected the gaps between vehicles from observed traffic, with a minimum of 1.5 m used for fully congested traffic. A truncated exponential distribution with a lower bound of 5 m and upper bound of 10 m was used for the gaps in congested traffic in Croce and Salvatore (1998, 2001). Vehicle speeds are used as the governing function in calculating gaps in other models. Ivy et al. (1953) relate gaps to truck speed for long-span bridges, and for shorter spans the spacing between trucks is reduced to a constant 8 ft (2.44 m). In Buckland (1981) the gap is a function of speed, with a minimum gap of 1.5 m for stationary vehicles. In Vrouwenvelder and Waarts (1993) the gaps between stationary vehicles range from 1 to 5 m, calculated randomly, and from 4 to 10 m in congested flow, based on the vehicles' velocity. Other authors allow gaps to vary as the vehicles traverse a bridge. In Crespo-Minguillon and Casas (1997) an equation for passing vehicles over a bridge was derived based on the difference in velocity of successive vehicles, with 1.0 m the minimum bumper-to-bumper gap. Chen and Wu (2011) used a microsimulation model based on the Nagel-Schreckenberg (1992) cellular automata model with the length of each cell, and also the minimum distance between the centres of two successive vehicles, set as 7.5 m. OBrien et al. (2010) used commercial microsimulation software, Paramics, which employs a car-following and lane changing model, and calibrated the results using observed driver behaviour from video recordings of traffic on a bridge in the Netherlands. Caprani (2012) uses traffic microsimulation to calibrate normally-distributed gaps in a truck-only traffic stream that replicates the results of different traffic compositions and flow rates.

Once a short run of traffic simulation has been carried out and traffic loading has been calculated, a method of extrapolating these results to a desired return period is usually required. Buckland (1981) used the Gumbel extreme-value distribution of 3-month maxima to predict the maximum shear or bending moment for any return period. Croce and Salvatore (1998, 2001) calculated an analytical expression for the cumulative distribution function of the maximum load effect caused by vehicle convoys over a given time interval. From this an equivalent uniformly-distributed load (EUDL) was

generated that represented the characteristic effects for a return period of 1000 years. In Flint and Neill (1986), characteristic values of load effects were derived from both normal and extremal distributions. For Vrouwenvelder and Waarts (1993) the distributed lane loads were derived and found to be normally distributed, and were thus extrapolated for comparison with the Eurocode traffic model. In Crespo-Minguillon and Casas (1997) an extrapolation based on a Generalised Pareto Distribution fitted to the tail of the maximum one-week load effects was used to calculate traffic loading for long return periods. OBrien et al. (2010) extrapolated 1-hour maximum bending moments for 11 days to characteristic loads using Gumbel probability paper. In Cremona and Caracilli (1998), optimal fittings of Rice's formula, through a series of statistical tests (such as the Kolmogorov K -test), from the tail of the level crossings histograms obtained from the combination of WIM records and influence surfaces allowed the authors to extrapolate the load effects to any return period. In Dietlevsen and Madsen (1997) formulas were derived for the mean and the intensity of the white-noise traffic-load field, and the white-noise Gaussian model was used to calculate the maximum load effects for a given return period. More recently, Soriano et al. (2016) presented a simplified approach for extrapolation based on a normal tail, while OBrien et al. (2015b) apply and compare alternative approaches.

In more recent research, traffic is typically recorded onsite in a free-flowing state using WIM devices, as the recorded traffic data is less reliable at lower congested traffic speeds (Klein et al. 2006). Congested traffic is considered to govern the maximum load effects for long-span bridges and previous research made assumptions regarding the vehicle arrival process, inter-vehicle gaps and traffic speed based on free-flowing traffic. Further work has shown that the vehicle arrival process for congested traffic is different to free-flowing traffic. Car drivers tend to move to the fast lane during the breakdown from free-flowing to congested traffic, resulting in a higher percentage of truck platoons in congested traffic compared with free-flowing traffic (OBrien et al. 2010, Caprani et al. 2012a, Enright et al. 2012). These truck platoons have a significant impact on traffic loading on long-

span bridges (Enright et al. 2012) and are hence important factors in the accurate calculation of traffic loading.

Full traffic microsimulation captures truck platooning and also avoids the need to make assumptions regarding inter-vehicle gaps and vehicle speed in congested traffic. It is a well-established and validated technique for modelling congested traffic (Treiber et al. 2001a, 2001b, Helbing et al. 2002). However, traffic microsimulation is computationally intensive and hence long-run simulations required for calculating lifetime maximum traffic loading effects, equivalent to a return period of 1000 years, are not presently achievable in an acceptable timeframe.

1.3 Outline of this work

To address the computational-demand problem with using traffic microsimulation to estimate traffic loading for long return periods, a ‘pseudo-microsimulation of congested traffic’ (PMCT) model is proposed in this work. It is validated using six months of free-flow WIM data recordings, including cars and other light vehicles. This data serves as the input to a full traffic microsimulation model which has the ability to reproduce complex driver phenomena based on driver interactions. The free-flowing traffic in the WIM data is passed through the microsimulation which, through the use of ‘virtual’ bottlenecks, causes congested traffic.

It is necessary to identify the significant characteristics of congested traffic to ensure that the proposed PMCT model reproduces these characteristics. The PMCT model should generate a continuous stream of congested traffic in a much more computationally-efficient manner than a full traffic microsimulation model. It should be noted that the proposed model is validated against the full microsimulation only, and not directly against any recorded congested traffic. Indeed, as may be seen from the literature review, there is no suitable directly-recorded congested traffic data available (for technical reasons to do with the capabilities of traffic sensors). Nevertheless, should such data become available in the future, it can be used directly in the PMCT framework, without recourse to a

microsimulation. In the meantime, the parameters of the microsimulation can be tuned to the traffic characteristics of the site of interest (Kesting and Treiber 2008). The key characteristics which are studied in detail and incorporated in the new model are:

- Vehicle occurrence: in particular, modelling the platooning behaviour of trucks and cars which is evident in congested traffic.
- Vehicle speeds.
- Speed-gap relationship (where gap is defined here as the bumper-to-bumper gap, which can be measured in terms of time or length).

2 Basis of work

2.1 Traffic data

The WIM data used for this study was obtained from the A4 (E40) at Wroclaw, Poland. In total over 22 weeks of traffic was recorded from 1 January to 5 June 2008. Essential features of this data set are that it includes cars, which are typically removed from most WIM data during roadside processing, rendering such data inapplicable for modelling congestion, and that vehicle arrival times were recorded to the nearest one hundredth of a second. For operational reasons, measurements on some days were not continuous, leaving 87 days of weekday traffic which is suitable for further use. Both lanes of traffic in one direction were measured. Weekends exhibit quite different traffic composition, and so are excluded, as is common in similar studies. Trucks with up to a maximum of nine axles were observed. Figure 1 shows the average hourly traffic flows, percentage of trucks and truck GVW from the site data. In this work, trucks are defined as any vehicle with a GVW over 3.5 t with all other vehicles defined as cars.

Figure 1. Traffic statistics from Wroclaw WIM data.

Vehicle overhangs (distance from the front and rear of the vehicle to the front and rear axles respectively) were not present in the WIM data and this makes the modelling of bumper-to-bumper gaps more difficult. A database of vehicle dimensions was compiled from information supplied by European vehicle manufacturers. Over 1000 vehicles are included in this database. These vehicle dimensions, along with other published data (Page and Ricketts 1997, Cremona and Carracilli 1998), were used to categorize vehicles by axle configuration and front and rear overhangs were added to the WIM data based on each vehicle's axle configuration. From Figure 2, it can be observed that the front overhang of two-axle trucks remains almost constant and the back overhang increases, both with increasing wheelbase. This relationship is similar for all trucks with the front overhang typically in the region of 1.3 to 1.4 m and the rear overhang varying between 0.8 to 4.3 m.

Figure 2. Two-axle Truck Vehicle Overhang versus Wheelbase (length from front axle to back axle of a vehicle).

2.2 Traffic microsimulation

Traffic microsimulation is used here to induce congestion in the free-flowing traffic recorded in the WIM data. A computer program, *Simba* – *Simulation for Bridge Assessment* (Caprani and OBrien 2008, Caprani 2010, Caprani et al 2016), based on the Intelligent Driver Model (IDM) developed by Treiber and others (Treiber et al. 2000a, 2000b) is used in this work. The model is based on a continuous function of an acceleration and deceleration component:

$$\frac{dv(t)}{dt} = a \left[1 - \left(\frac{v}{v_0} \right)^\delta - \left(\frac{s^*(v, \Delta v)}{s} \right)^2 \right] \quad (1)$$

where: v is the velocity of the current vehicle, v_0 is its desired velocity, δ is the acceleration component, s^* is the desired gap (see Equation (2)), s is the actual gap to the vehicle in front, and Δv is the velocity difference (approaching rate) to the lead vehicle. The desired gap is calculated using:

$$s^*(v, \Delta v) = s_0 + s_1 \sqrt{\frac{v}{v_0}} + T v + \frac{v \Delta v}{2\sqrt{ab}} \quad (2)$$

where: s_0 is the minimum jam distance, s_1 is the elastic jam distance, T is the safe time headway,

a is the maximum acceleration and b is the comfortable deceleration. To allow for lane changing, the model developed by Kesting et al. (2007) is used. This model, MOBIL (Minimizing Overall Braking Induced by Lane changes), is based on two criteria: (1) Incentive criterion – where the driver considering changing lane weighs up the advantages for the proposed change, and (2) Safety criterion – where the lane-changing operation the driver is considering must be safe (Caprani et al. 2012). The IDM parameters used in this work, listed in Table 1, are taken from Helbing et al. (2002) (parameters 1-7) and Kesting et al. (2007) (parameters 8-10), and are similar to other such work (Caprani et al. 2016, Treiber et al. 2000a).

Table 1. IDM parameter values used in the microsimulation model.

Parameter	Car	Truck
1. Safe time headway, T (s)	1.2	1.7
2. Maximum Acceleration, a (m/s^2)	0.8	0.4
3. Comfortable deceleration, b (m/s^2)	1.25	0.8
4. Minimum jam distance, s_0 (m)	1	1
5. Elastic jam distance, s_1 (m)	10	10
6. Desired Velocity, v_0 (km/h)	120 (± 20)	80 (± 20)
7. Acceleration exponent, δ	4	4
8. Lane change politeness factor, p	0.25	0.25
9. Outside lane bias factor (m/s^2)	0.3	0.3
10. Lane change threshold (m/s^2)	0.1	0.1

In this research, the WIM traffic data, with the fitted overhangs, is split into “Day” traffic (06:00-22:00) and “Night” traffic (22:00-06:00). This is done because the traffic composition (percentage of trucks and cars) and flow are quite different in the two periods, as can be seen in Figure 1. The traffic flow during the day is approximately 4.5 times that of the night traffic, and congested traffic is therefore more likely to occur during the day. However, the higher proportions of trucks travelling at night may increase the likelihood of multiple trucks being present on a bridge at a given time with

consequent more severe traffic load effect. Thus, the two types of traffic are treated separately here, with the proposed model calibrated to each traffic stream to illustrate its applicability to very different types of traffic.

A virtual two-lane road, 21 km in length with open boundaries, is used for the microsimulation. To cause congestion on the virtual road, flow restricting road features (bottlenecks), are added between 19.5 and 20.0 km. These features consist of a speed limit, which reduces each vehicle's desired speed to 20 km/h, and a gradient, which increases drivers' safe time headway by 0.5 s. The aim is to induce homogeneous congested traffic (HCT) which is characterized by a low average velocity, but in contrast to full stop traffic, has a relatively high traffic flow (Helbing et al. 2002). OBrien et al. (2015) found that either full stop traffic (bumper-to-bumper traffic queue) or HCT produced greatest traffic load effects for long-span bridges, depending on the percentage of trucks and the bridge length. Full stop traffic events tend to occur less frequently (Schönhof and Helbing 2007, Tasnim et al. 2008, Giuliano 1989, Skabardonis et al. 1997, Skabardonis et al. 1999, Rodgers et al. 2006) and are typically caused by exceptional events leading to all lanes being closed. In this research only HCT is considered.

Figure 3 shows the spatio-temporal density plot of one block of Day traffic. A severe bottleneck strength (i.e. flow capacity reduction) of approximately 275 veh/hr/lane induces HCT, which builds at the bottleneck and moves downstream for the first 14 hours of the traffic block.

Figure 3. Spatio-temporal density plot of one block of Day traffic for two lanes.

The traffic flow for an entire measured Day typically produces 8.9 hours of congested traffic, while the traffic over an entire Night block produces 1.75 hours. In the microsimulation model, virtual bridges with lengths of 100 m, 200 m, 500 m, 1000 m and 2000 m begin at 17.25 km on the virtual road. The total load caused by congested traffic on each bridge at a given time is calculated and the maximum load for every two-hour period is obtained. The duration of two-hours is taken to represent two rush hour periods per day, of full congestion – morning and evening. It is a common choice (e.g. OBrien et al 2015a, Caprani et al 2016) and is quite onerous. For different durations of congestion per day, the return period can be adjusted accordingly.

2.3 Overview of pseudo-microsimulation of congested traffic (PMCT) model

The PMCT model proposed here replicates the important traffic characteristics that affect traffic loading, and makes it possible to conduct long-run simulations in an acceptable time-frame. Figure 4 gives an overview of how the model is implemented.

Figure 4. Pseudo-microsimulation of congested traffic (PMCT) model overview.

The steps described for implementing the PMCT model for a particular bridge are:

1. Obtain on-site WIM traffic data (which will be free-flow).
2. Add vehicle overhangs based on axle configuration, if necessary.
3. Process the recorded traffic through microsimulation, using a bottleneck to cause congestion, for a relatively short period (e.g. 20 months).
4. Derive key traffic characteristics from the microsimulation output:
 - Vehicle occurrence – platoon distribution and vehicle transition probability matrix;
 - Speed-time relationship;
 - Speed-gap relationship.

Using these key characteristics, the PMCT model can generate many years of congested traffic which can be passed over bridge influence lines, yielding the required load effects. Typically time steps of 0.2 seconds are used in moving the traffic over the bridge. Two lanes of traffic in the same direction are generated for this work, although the model is not restricted to two lanes. The composition of the

traffic stream (trucks and cars) is based on a vehicle occurrence model using a platoon distribution and transition probability matrix (TPM). Vehicle characteristics (weight, axle configuration) are taken by sampling (or ‘bootstrapping’) from the substantial set of recorded vehicles in the WIM data. At each time interval, the position of each vehicle in the stream is updated using its current speed and acceleration, obtained from the speed-time and speed-gap relationship. The key difference between this approach and full microsimulation is that only the fully-congested traffic on the bridge is modelled, without the need to model a much longer road length which includes the transition from free-flowing to congested traffic. The daily or yearly maximum values of the load effect of interest are recorded and plotted on Gumbel probability paper to obtain the characteristic loading.

3 Vehicle occurrence modelling

3.1 Vehicle platooning

A stream of traffic in one lane may be characterised as a sequence of truck and car platoons, where a platoon is a group of vehicles of the same type travelling together. While a ‘platoon’ may consist of just a single vehicle, it is the occurrence of large platoons of trucks which is of interest for bridge loading. If a purely random arrival process is used to generate streams of trucks and cars, platoons of varying lengths will undoubtedly form. However, an examination of the WIM data for free-flowing traffic and of the congested traffic produced by microsimulation shows that the formation of platoons is not purely random. This is illustrated in Figure 5 for congested traffic obtained from traffic microsimulation, and uses conditional probabilities to show that the probability of the next vehicle in a stream of traffic being a car or truck increases with the number of vehicles of the same type travelling in front. For example, $P(T|T_i)$ is the probability that the next vehicle is a truck, given that there is a truck platoon containing i trucks in front of it, and this can be seen to increase with increasing values of i . Conversely, the probability $P(C|T_i)$ that the next vehicle is a car, given that there is a truck platoon of length i in front is found to decrease with increasing i .

Figure 5. Platoon dependency plot for congested traffic (from microsimulation) – single lane platoons on two lane traffic.

While platoons occur in free-flowing traffic, they tend to become even more pronounced in the break down from free-flow to congested (Enright et al. 2012). To investigate the cause of platooning in congested traffic, different mechanisms were simulated, and it was found that the observed platooning behaviour can be accurately modelled by assuming that car drivers decide to change lane probabilistically based on a set of criteria. In particular car drivers show an increased probability of changing lane as the number of trucks travelling in front of the car increases (Enright et al. 2012). It has previously been found that this platooning behaviour has a significant effect on maximum lifetime loading on long-span bridges (Enright et al. 2012), and so any proposed model must reproduce this behaviour. Consequently, in this work, cumulative distribution functions (CDFs) of car and truck platoon lengths are calculated for the congested traffic output from microsimulation. Figure 6 shows these CDFs for the micro-simulated congested Day traffic, and also shows that they are significantly different from the corresponding CDFs for purely random vehicle occurrence.

Figure 6. Platoon length distributions: comparison of microsimulation congested vehicle arrival and purely random vehicle arrival (each with the same proportion of trucks/cars - lane 1 Day Traffic).

The CDFs of Figure 6 are strongly dependent on the overall percentage of trucks in the traffic stream, with an increasing probability of large truck platoons occurring as the percentage of trucks increases. In the Day traffic at the site in Poland, the percentage of trucks remains relatively constant around 34%, whereas in Night traffic it ranges from an average of 33% trucks from 22:00-23:00 to 75% trucks from 03:00-04:00. Due to this variation in traffic composition during each hourly block in the Night traffic, the distribution of truck platoon lengths for Night traffic is non-stationary. To account for this, it is necessary to calculate truck/car platoon length CDFs for different truck percentages, and satisfactory results are found using six different truck/car platoon length CDFs, with each CDF created from 1.5 hour blocks, covering the truck percentage range from 33% to 75%.

The truck/car platoon length CDFs described above are used in the PMCT model to determine the length of successive platoons of trucks and cars. Within each truck platoon, it is still necessary to represent each truck by assigning a number of axles, and a GVW, amongst other properties. To account for the observed tendency of, for example, 5-axle trucks to travel together within a platoon, a TPM ('Vehicle Sequence TPM') is calculated for each lane from the congested traffic output from the microsimulation. This measures the probability of a j -axle truck following an i -axle truck, and

also of a j -axle truck following a car (i.e. the type of truck likely to be leading a platoon). Using the Vehicle Sequence TPM, the next truck or car in the platoon is sampled from the WIM vehicle database recorded at the site. This vehicle and platoon occurrence model has previously been found to accurately replicate the truck and car platoon lengths found in the congested traffic produced by microsimulation, while also maintaining the correct overall ratio of cars to trucks (Enright et al. 2012).

3.2 Effect of vehicle platooning on traffic loading (simple arrival model)

To demonstrate the importance of vehicle platooning for bridge loading, a comparison is carried out where one traffic stream is generated using a purely random traffic arrival process, and another traffic stream is generated using the proposed platoon arrival model incorporating the transition probability matrix for truck types. Simulations are carried out to model the Day and Night traffic separately due to their differing characteristics (see Figure 1). Both traffic streams are passed across two-lane bridges of different lengths, with a constant (static) inter-vehicle gap and traffic speed for simplicity. The maximum traffic load that occurs in each two hours of congested traffic is recorded. The results from these simulations are extrapolated to 10 and 1000-year return periods – the rationale for which will be explained later in Section 5.1 – using the Generalised Extreme Value distribution (see Section 5.1) and the results are shown in Figure 7 for the two simulations for a 100 m bridge using Day traffic. The purely random arrival process can be seen to significantly underestimate the extrapolated total load results compared with the platoon arrival process.

Figure 7. Periodic maximum load effects from purely random vehicle arrival process compared with the platoon traffic (see text) for 100 m two-lane bridges (Day traffic).

Figure 8 shows the difference in extrapolated total load given by the two arrival processes for a number of bridge lengths. The influence of the arrival process decreases with increasing bridge length, with up to 28% underestimation in results for a 100 m bridge compared to a 5% difference for a 2000 m bridge. This suggests that the overall percentage of trucks on the bridge becomes more important than the spatial layout of trucks in the traffic stream for increasingly longer loaded lengths. For short-span bridges, the spatial layout of trucks is very important as far fewer vehicles will fit on the bridge at the same time.

Figure 8. Extrapolated load effects from purely random vehicle arrival process compared with the platoon traffic for two-lane bridges of different lengths.

4 Vehicle speed and gap modelling

4.1 Congested traffic speed-gap relationship

As congested traffic crosses a long-span bridge, vehicle speeds and inter-vehicle gaps will vary. Traffic theory suggests that the relationship between traffic speed and inter-vehicle gaps is asymmetric with respect to speed, with a hysteresis loop often used to describe the relationship (Newell 1965, Forbes 1963, Treiterer and Myers 1974, Maes 1979, Zhang 1999). The speed changes in these works represent sudden or large decelerations/accelerations ‘shocks’, which indicate a transition from free flow to congested flow or vice-versa (Wang et al. 2005, Chen 2012). This phenomenon is implicitly replicated in the microsimulation model (Treiber et al. 2000b). However, the speed-gap relationship of relevance to this work is the one that applies in homogeneous congested traffic (HCT) where traffic has relatively small accelerations and decelerations (under 0.2 m/s). For this type of traffic, the present work has found the speed-gap relationship to be approximately linear, with no hysteresis loop, as shown in Figure 9. It can be seen that the speed-gap relationship is different between different types of vehicles due to the different acceleration/deceleration rates applicable to the different type of vehicles, as is previously noted in the literature (Aghabayk et al. 2012). This is also evident in Table 2. To account for this, the speed-gap relationship is split into four types (car following car, car following truck, truck following truck and truck following car). This speed-gap relationship is used to time the introduction of each new vehicle into the simulated traffic stream.

Table 2. Average time gaps between different vehicle types in homogeneous congested traffic.

Vehicle Sequence	Time Gap (s)
Car following car	2.7
Car following truck	3.7
Truck following truck	3.9
Truck following car	2.9

Figure 9. Speed-gap relationship taken from traffic microsimulation for two vehicle combinations (Day traffic, lane 1).

4.2 Congested traffic speed-time relationship

The speed of vehicles passing a particular point can be represented by a time series, as shown by Figure 10 for congested traffic in the microsimulation. It can be seen that the speed-time series contains periods where the traffic is travelling at a constant speed, described here as “Constant Speed Phases” (CSPs). Due to the presence of CSPs, the speed-time series is non-stationary and an autoregressive time series model, such as ARIMA is not suitable for modelling purposes (Box et al. 2008). During the times when the speed is in a CSP the gaps between vehicles are much less variable, as shown in Figure 11, with the average gap in a CSP 10% lower than at other times. The average CSP speed is 7% lower than the overall average congested speed, but the speed-gap relationship in

Figure 9 is still valid for CSPs.

Figure 10. Speed- and gap- time series from the microsimulation showing Constant Speed Phase (CSP).

Figure 11. Gap distribution for different speed phases, non-CSP and CSP.

The proposed speed-time model is based on the difference in speed between successive vehicles as a function of the speed of the leading vehicle. This bivariate relationship, obtained from the microsimulation, is shown in Figure 12. A positive change in speed indicates that the following vehicle is travelling faster than the leading vehicle, thus closing the gap between the two. It can be seen that following vehicles are more likely to close the gap to the leading vehicle when the speed of

the leading vehicle is low, and to widen the gap when the speed of the leading vehicle is higher. If the leading vehicle is travelling at constant speed (i.e. in a CSP), the following vehicle is also likely to be travelling at constant speed. If the leading vehicle's speed is low and it has begun to accelerate, the probability the following vehicle will also accelerate is high. The opposite is also true at the beginning of a deceleration phase at higher speeds. This tendency for vehicles to stay in phase is modelled here using a Markov chain. Based on the speed and phase of the leading vehicle, the following vehicle's speed and phase is estimated probabilistically.

Figure 12. Speed relationship between successive vehicles, from the slow lane of traffic microsimulation.

To recreate the flow of traffic traversing a bridge using the PMCT model, a number of metrics are required from the congested traffic. As noted earlier, for the present work, microsimulation is used to obtain these metrics since the WIM data used is for free-flow traffic (as is typical). For sites where the required congested traffic data can be obtained directly, microsimulation is not required. The metrics are:

1. Three separate TPMs for each possible phase (acceleration/CSP/deceleration) of the lead vehicle ('Phase TPM') - where the 'TPM phase' is the lead vehicle's speed, and the 'TPM transition' is the following vehicle's phase.

2. Two separate TPMs for the acceleration/deceleration phases of the following vehicle ('Change in Speed TPM') – where the 'TPM phase' is the lead vehicle's speed, and the 'TPM transition' is the following vehicle's speed.
3. One TPM ('CSP Duration TPM') – where the 'TPM phase' is the following vehicle's speed, and the 'TPM transition' is the length of time the traffic will travel at the following vehicle's speed.
4. Four separate speed-gap functions based on the type of lead/following vehicle (see Section 4.1).

In the PMCT model, at all times each vehicle has a current speed, a 'next' speed, and a time remaining in a CSP phase (if a vehicle is not in a CSP phase the time remaining is zero). When the leading vehicle in the traffic stream reaches its 'next' speed, a new 'next' speed for all the vehicles in that lane is calculated. To calculate the new 'next speed' for a vehicle, the vehicle's phase (acceleration/CSP/deceleration) is found based on the speed and phase of the vehicle in front. The 'Phase TPM' is used to determine the following vehicle's phase, h :

$$h_{i+1} = \Pr(h | h_i, v_i) \quad (3)$$

where h_{i+1} is the phase of the following vehicle, h_i is the phase of the leading vehicle and v_i is the speed of the leading vehicle.

To determine the following vehicle's speed the 'Change in Speed TPM' is used. Based on the leading vehicle's speed and the following vehicle's phase, as calculated in Equation 3, the transition probability can be used to determine the speed of the following vehicle:

$$v_i + \Delta v_{i+1} = \Pr(\Delta v | h_i, v_i) \quad (4)$$

where Δv_{i+1} is the change in relative speed between the leading vehicle and the following vehicle. For the first vehicle in a simulation, the vehicle's current speed and phase are taken as h_i and v_i for Equations (3) and (4). If the following vehicle enters a CSP, a TPM ('CSP Duration TPM') is used to calculate the length of time the traffic will travel at the following vehicle's speed. Based on the following vehicle's speed, as calculated in Equation (4), the transition probability can be used to determine the CSP duration:

$$T_{CSP} = \Pr(T | v) \quad (5)$$

where T is the length of time the traffic will remain in a CSP phase and v is the following vehicle's speed. Based on the length of time the traffic will be in a CSP phase, the number of following vehicles and the length of time each of these vehicles will be in a CSP phase is calculated and recorded against each vehicle.

The time taken for each vehicle to reach its 'next' speed is calculated using the current speed, 'next' speed and inter-vehicle gap (calculated using the speed-gap relationship). Using the speed-gap model described, the traffic crosses the bridge (in 0.2 s time steps) with varying speeds and inter-vehicle gaps which are statistically based on the congested traffic observed in the microsimulation. This method was found to accurately model the speed-time series and gap distributions measured in the microsimulation output (Table 3), and has the ability to generate long streams of congested traffic efficiently.

Table 3. Comparison of speed and gaps statistics for vehicles entering bridge – microsimulation vs. PMCT model.

		Maximum	Minimum	Mean	Standard Deviation
Speed (km/h)	Microsimulation	7.56	3.24	4.27	0.40
	PMCT Model	7.56	3.24	4.28	0.42
Gap (m)	Microsimulation	19.14	4.02	7.25	2.43
	PMCT Model	19.13	3.86	7.24	2.45

Note: Gaps are measured from back axle of lead vehicle to front axle of following vehicle)

4.3 Effect of speed-gap model on traffic load effect (simple gap model)

Some previous methods of calculating bridge traffic loading used a constant gap between vehicles and passed vehicles across the bridge at a constant speed. The proposed PMCT model, in seeking to replicate microsimulation, allows each vehicle's speed, and consequently inter-vehicle gaps, to vary as traffic crosses a bridge. To test the importance of this aspect for traffic loading, two simulations were carried out for the same congested traffic streams. In the first simulation, vehicles are passed across bridges of different lengths with constant inter-vehicle gaps and speeds (using the average values from the microsimulation). For the second simulation, the same traffic is passed across the bridges using the proposed speed-gap model, allowing the vehicle speeds and inter-vehicle gaps to vary across the bridge. The resulting load effects are extrapolated from 20 months of simulation to 10 and 1000-year return periods (see section 5.1). It can be seen in Figure 13 that the constant gap and speed model underestimates the extrapolated load effect values. This underestimation seems to be independent of the bridge length but is significant, with up to 14% underestimation in total for a single lane 100 m bridge for the 1000-year return values. To explain this, we note that when inter-vehicle gaps vary there will be occasions when they are less than the overall average gap used in the constant inter-vehicle gaps simulation. This situation then has a greater number of axles present on the bridge, and consequently larger load effects which affect the extreme. As both simulations have the same average gap, it is clear that allowing the inter-vehicle gaps to vary as the traffic crosses the

virtual bridge is an important factor to consider in modelling long-span bridge loading (a similar finding to Lipari et al 2010).

Figure 13. Effect of constant (static) and varying (dynamic) gaps on extrapolated total load for various bridge lengths (single-lane bridge).

5 Results from PMCT model

5.1 Comparison of proposed model with full microsimulation

To assess the accuracy of the proposed model for bridge loading, load effects calculated from microsimulation are compared with those from the PMCT model for different bridge lengths. For the purposes of comparison, it is assumed that two hours of congestion occur every day, and both models are used to simulate 125 two-hour ‘days’ of traffic. It is further assumed that there are 250 such ‘economic’ days per year (excluding weekends and holidays). The Generalized Extreme Value distribution (GEV), is used to extrapolate the two-hour block maximum values from both simulations. The GEV distribution is (Coles 2001):

$$G(z) = \exp \left\{ - \left[1 - \xi \left(\frac{z - \mu}{\sigma} \right) \right]_+^{1/\xi} \right\} \quad (6)$$

where $[h]_+ = \max(h, 0)$ and μ, σ, ξ are the location, scale and shape parameters respectively.

The results from the simulations are extrapolated to a 1000-year return period (which is the value with approximately 5% probability of exceedance in 50 years) as used in the Eurocode (2003) for design, and a 10-year return period which is considered as one suitable for bridge assessment (Dawe 2003). Results for the total load on the bridge are plotted on Gumbel probability paper for a 2000 m two-lane bridge (for Night traffic) in Figure 14. Comparisons of extrapolated values for different bridge lengths and types of traffic are shown in Figure 15, and it can be seen that the PMCT model accurately reflects the results from the microsimulation for a range of bridge lengths, with percentage errors between +4% to -7%.

Figure 14. Comparison of total load from microsimulation and PMCT for 2000 m two-lane bridge (Night traffic).

Figure 15. Comparison of extrapolated total load from microsimulation and proposed PMCT for two-lane bridges of different length.

The observed close match between simulation approaches for total load on the bridge indicates that the same vehicle weights are present in both models for a given bridge length. However, using the total load does not prove that the distribution of vehicles on a given bridge length is similar in both models. To further test how well the proposed model replicates the bridge loading from the full microsimulation, more complex influence lines are considered. Using finite element analysis, influence lines are generated for two bridges: a cable-stayed bridge – the Sidney Lanier Bridge in Brunswick, GA, and a suspension bridge – the Golden Gate Bridge in San Francisco, CA. A detailed description of these influence lines is given in Enright et al. (2013). Following the findings of Ruan et al (2016), it is taken that the live load effects can be assumed linear when considered alone, in contrast to the geometrically non-linear response of large cable-supported bridges to dead load. The influence lines considered for both bridges are (Figures 16a and 15b):

- LE1. Axial force in one of the towers (pylons) (R_A) ;
- LE2. Bending moment at the foot of one of the towers (pylons) (M_A);

For the Golden Gate bridge:

- LE3. Bending moment in the deck at mid-span (M_B);
- LE4. Tension force in the main cable (T_1).

For the Sidney Lanier bridge:

- LE3. Tension force in Cable 1 (T_1);
- LE4. Tension force in Cable 2 (T_2).

Figure 16a. The Sidney Lanier Bridge: (a) elevation and (b) influence lines (see the main text for definitions of the terms).

Figure 16b. The Golden Gate Bridge: (a) elevation and (b) influence lines (see the main text for definitions of the terms).

The Day traffic from the Wrocław site is used as the basis for generating congested traffic on both bridges. In both the full microsimulation and in the PMCT model, it is again assumed that two hours of congestion occurs per day, and maximum load effects are calculated for each 2-hour block of congested traffic. These daily maximum load effects are extrapolated as before to get the characteristic load effects. In Figure 17 extrapolated 10-year and 1000-year load effect results for

each of the influence lines from both the microsimulation and the proposed model are compared. The proposed model results match the results from the microsimulation within $\pm 5\%$ for both bridges.

Figure 17. Ratios of extrapolated (10-year and 1000-year) load effects from the microsimulation and the PMCT for different influence lines (Note: GGB=Golden Gate Bridge; SLB=Sidney Lanier Bridge).

5.2 Simulating 1000 years of traffic

The main aim of the PMCT model is to replicate the important aspects of the microsimulation in a much more computationally-efficient manner. To simulate 1000 years of congested traffic, the microsimulation program used here would take approximately 90 years on a current standard personal computer. The model presented here can simulate 1000 years of congested traffic in just 72 hours – a long but acceptable duration which could be further shortened using parallel computation, for example. Figure 18 shows the average annual maximum total load for a two-lane 2000 m bridge (Day and Night traffic) from ten 1000-year simulations. The large difference between the load effects for Day and Night traffic evident in Figure 18 is explained by the much higher percentage of trucks in Night traffic which leads to the formation of much longer truck platoons in congested traffic. The same simplified assumption of two hours of congestion per day is applied to both types of traffic. As the traffic flow during the Day traffic is 4.5 times that of the Night traffic, it is more probable that

congestion will occur during the Day traffic. However, a crash or traffic abnormality may lead to congestion during the Night traffic and the bridge in question needs to be able to cope with the associated traffic load (Lipari et al. 2012). Further research into the occurrence rate of congestion for different traffic volumes is required and this work merely models the recorded ‘Day’ and ‘Night’ traffic as two different types of traffic.

Figure 18. Annual total load from Day traffic and Night traffic 1000-year PMCT simulation for two-lane 2000 m bridge.

5.3 Comparison between extrapolated and 1000- year simulated load effect results

There are uncertainties associated with extrapolating short run results to longer return periods. With computationally intensive methods, it might be feasible to simulate six months of traffic and then extrapolate the results to a 1000-year return period (as described in section 5.1 and shown in Figure 14). With the proposed model, it is possible to simulate 1000 years of traffic directly, thus avoiding the need for extrapolation. This is shown in Figure 19 where ten PMCT 1000-year simulations and the extrapolated 1000-year total load results are compared. It can be seen that the extrapolated results underestimate the total load by up to 33% (for Day traffic) compared with the average 1000-year results. The differences for Night traffic are much smaller than for Day traffic. These differences are

explained by the fact that extremely heavy trucks (up to 91.6 t) were recorded during the day, and during the short-run microsimulation these heavier trucks do not appear in platoons, as the percentage of trucks in Day traffic is relatively low (Figure 1). However, during the 1000-year PMCT these heavy trucks were found to occur in truck platoons and these events lead to much higher load effects than occurred in the microsimulation. In Night traffic the percentage of trucks is high (see Figure 1) and therefore large truck platoons, including heavier trucks, occur frequently in the short-run microsimulation.

Figure 19. Total load from 1000-year PMCT simulation and extrapolated from 6 months of microsimulation (two-lane bridge).

Ten PMCT 1000-year simulation were also run with the proposed model for Day traffic using the influence lines for the Sidney Lanier Bridge and the Golden Gate Bridge. Yearly load effect maxima were captured for each load effect considered and compared to the extrapolated load effect results from the microsimulation, see Figure 20.

Figure 20. 1000-year load effect ratios from the microsimulation (extrapolated) and the PMCT (simulated) for different influence lines (Note: GGB=Golden Gate Bridge; SLB=Sidney Lanier Bridge).

As with the total load comparison, the simulated results from the proposed model are significantly larger than the extrapolated results from the microsimulation. The largest difference is for the average Load Effect 2 (bending moment at the foot of one of the towers) from the Sidney Lanier Bridge which is 84% greater from the proposed model. This result shows that it is important to use as much traffic data obtained from the site as possible, to reduce extrapolation errors.

5.4 Comparison between model and design code characteristic loading

Results from the proposed model are compared with corresponding values in different design codes in Figure 21. The EUDL is calculated for Eurocode LM1 (2003), British Standard 5400 HA loading (2006) and the AASHTO LRFD Bridge Design Specifications HL93 loading (2007) and compared with the results from the proposed model for Day and Night traffic. The Eurocode LM1 is intended for bridges up to 200 m, the BS 5400 HA loading applies up to 1600 m, and only recently was the AASHTO load model extended to all bridge lengths. These design code EUDLs have been included in Figure 21. The EUDLs are unfactored for two-lane bridges of different lengths, but with dynamic load included. There is no dynamic load component incorporated in the EUDLs calculated from the model but, as the traffic is travelling at such a low speed, dynamic load is not considered to be

important. The model EUDL is found to be higher than that of the three codes shown here for a 100 m bridge but significantly lower than the Eurocode LM1 and BS 5400 HA loading for bridges of 500 m or above.

Figure 21. Equivalent unfactored UDLs for two lanes of traffic (Note that AASHTO HL93 has multilane factor of 1.2 applied).

6 Conclusions

The proposed PMCT model addresses the need for a more computationally tractable approach to modelling long-span bridge loading. It replicates the phenomena of congested traffic fundamental to bridge traffic loading. Vehicle characteristics from WIM recordings, with fitted overhangs, have been used as an input to a traffic microsimulation model in which congestion is induced through the use of a virtual bottleneck. From the output of the microsimulation, parameters important to traffic loading on bridges are identified. The tendency for vehicles of the same type (car or truck) to arrive successively is found to be of particular importance and the effect of this on traffic loading is demonstrated. Allowing vehicle speeds and inter-vehicle gaps to vary as traffic crosses a bridge is found to best replicate the traffic loading from the microsimulation. These parameters, along with others from the microsimulation, have been modelled and form the basis of a computationally efficient model which has the ability to carry out long-run simulations to obtain characteristic load

effects, for a return period of 1000 years, in an acceptable time frame. In doing so the model avoids the uncertainties associated with extrapolation techniques required in other models.

For the design of super-long next-generation bridges, and for the assessment of existing aging bridges, more refined site-specific loading models are required. Also, good knowledge of the significant contributing factors in traffic loading on long-span bridges is essential to avoid undue conservatism. This work presents an approach that better replicates those important factors of traffic loading on long span-bridges and in doing so gives greater confidence in the resulting characteristic load effects.

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