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# The use of micro-simulation for congested traffic load modelling of medium- and long-span bridges

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**ABSTRACT:** This paper presents a new approach to the modelling of congested traffic loading events on long span bridges. Conventional traffic load models are based on Weigh-In-Motion data of non-congested traffic, or something similar to a Poisson Arrival process. In neither case do they account for the mixing between lanes that takes place as traffic becomes congested. It is shown here that cars move out from between trucks as traffic slows down which results in a higher frequency of long platoons of trucks in the slow lane of the bridge. These longer platoons increase some characteristic load effects under the slow lane by a modest but significant amount. Micro-simulation, the process of modelling individual vehicles that is widely used in traffic modelling, is presented here as a means of predicting imposed traffic loading on long-span bridges more accurately. The traffic flow on a congested bridge is modelled using a random mixing process for trucks and cars in each lane, where each vehicle is modelled individually with driver behaviour parameters assigned randomly in a Monte Carlo process. Over a number of simulated kilometres, the vehicles move between lanes in simulated lane-changing manoeuvres. The algorithm was calibrated against video recordings of traffic on a bridge in the Netherlands. Extreme value statistics of measured strains on the bridge are then compared to the corresponding simulation statistics to validate the model. The micro-simulation algorithm shows that the histograms of truck platoon length are moderately affected by lane changing. This in turn is shown to influence some characteristic load effects of the bridge deck.

**Keywords:** micro-simulation; traffic load; bridge; loading; long; span.

## 1. Introduction

Whether for the assessment of existing bridges or the design of new ones, traffic load is generally represented in standards with notional load models. Such models are deemed to represent the worst loading condition in a specified return period – 1000 years in Eurocode 1 (EC1 2003). However, more accurate assessments of traffic loading can result in the retention of existing bridges for longer lives by proving that they are safe for the load carried. Furthermore, the capability to calculate characteristic traffic loading more accurately, opens up the prospect of designing longer spans than has been possible to date.

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A review of normal traffic loading on long-span bridges was carried out in by Buckland et al (1978) and Buckland (1981). The maximum load effects are found to occur when the traffic is stationary and 'bumper to bumper', with no dynamic amplification. The Flint and Neill Partnership (1986) carried out numerical Monte-Carlo simulations and statistical analysis of traffic loading models representative of traffic flows and mixes on long span bridges on a heavily trafficked commercial route, to determine characteristic traffic load effects when traffic is jammed. The characteristic values obtained were used to develop a load model consisting of an equivalent uniformly distributed load and a single knife edge load per lane. Due to the substantial increase in the proportion of heavy goods traffic since the 1970's, the design loadings obtained by the authors for long span bridge were considerably heavier than those recommended in the earlier code. The stochastic vehicle-queue-load model developed by Ditlevsen & Madsen (1994) is an analytical approach for the prediction of extreme load effects from any kind of traffic loads on long span bridges. In the model the traffic along the lanes is represented as 'white-noise' load fields. The mean and variance of the white-noise field depend on the traffic situation and are described by Gaussian random processes. The model covers the entire range of traffic from free Poissonian traffic to congested traffic with the full stop queue as a limit case. Nowak (1991 and 1999) developed Monte-Carlo live load models to perform studies on the effect of multiple vehicle presence for vehicles on a single-lane bridge and for vehicles in two adjacent lanes with various degrees of correlation between the truck weights. For a range of simply supported spans, a cumulative distribution function was determined for the traffic load effects considered. For longer spans the maximum effects were caused by the case of a fully correlated pair of trucks with headway distance of 5m that was associated with stationary or slowly-moving vehicles. The range of considered spans (from 12m to 60m), however, was not enough to draw conclusions for long-span bridges. Therefore, this study was only applicable for use in codes for moderate span bridges up to 60m.

This paper describes a new technique for the accurate modelling of traffic loading on long-span bridges. For multi-lane (same direction) traffic, a key challenge in assessing the extreme loading conditions is to model the movement of vehicles between lanes as traffic becomes congested. Driver behaviour under such conditions results in changes to the car-truck mix in each lane as car drivers seek to move out from behind trucks. This is important as it results in the formation of long platoons of heavy trucks in the slow lanes of the bridge. Previous methods, which seek to generate jams by reducing the gaps in free flowing traffic data, are inaccurate as they neglect the movements of cars out of lanes with large numbers of trucks and therefore underestimate the frequency of long platoons of trucks.

The approach adopted here is to model the behavioural response of vehicles to their surroundings using commercial micro-simulation software, Paramics. The Paramics microsimulation package ([www.sias.ie](http://www.sias.ie)) includes a microscopic car-following and lane-changing model, dynamic routeing and inclusion of intelligent transport systems. Its mathematical model is based on the psycho-physical theory developed by Weidemann (1974) and Fritzsche (1994). This model, calibrated using observed driver behaviour, is used to predict the mixing of multi-lane traffic on the bridge deck. A vehicle's response to its environment is described in the model by two parameters, driver's 'target headway' and 'reaction time'. The target headway is the bumper-to-bumper time gap that drivers try to achieve when following other vehicles in traffic. The reaction time is the lag in time between a change of speed of the preceding vehicle and the following vehicle's reaction to that change. Target headway and reaction time values are drawn from a statistical distribution representing the range of driver behaviour and randomly

allocated to the vehicles within the fleet. These parameters affect maximum speed and propensity to change lane of the individual vehicles.

In free-flowing traffic, vehicles attempt to travel at a speed closest to their target speed, allowing a safe distance from vehicles in front of them. For congested traffic, the vehicle is prevented from achieving its target speed so it considers a change into another lane if vehicles in that lane are travelling faster. The lane changing process itself considers a sequence of three steps: decision to attempt a lane change, choice of the target lane, and gap acceptance. In heavily congested traffic, when acceptable gaps are hard to find, a forced merging model is used to capture forced lane changing, i.e., where a driver applies pressure on others to provide the space needed in the target lane. Therefore, characteristic features of congested traffic such as stop-start behaviour, queuing and platoons of trucks occur naturally in the simulation for the same reason that they occur in the real world (Rapael et al 2000, Lieberman & Rathi 1999).

The methodology used to derive traffic load models for long-span bridges combines traffic micro-simulation with bridge finite element analysis. Vehicle weights and car-truck mix data in free-flowing traffic were collected from a weigh-in-motion (WIM) station. This is used to add statistically representative weights to the vehicles in the micro-simulation models and to set the initial conditions prior to the onset of congestion.

## **2. Site description, instrumentation and finite element model**

To validate the micro-simulation approach, a comparative study was carried out between the model and results measured on a 100m simply supported bridge span. The Moerdijk Bridge in the Netherlands spans the Hollands Diep River and forms a section of the A16 highway between Dordrecht and Breda. It is subject to congested traffic on a daily basis involving significant numbers of trucks. The complete bridge consists of ten simply supported spans of 100m and is a box girder with orthotropic stiffened steel plates of 10, 12 and 14mm and mastic asphalt surfacing of 60mm (Figure 1). It carries 3 lanes of traffic in each direction and serves more than two million vehicles annually, up to 14% of which is heavy truck traffic.

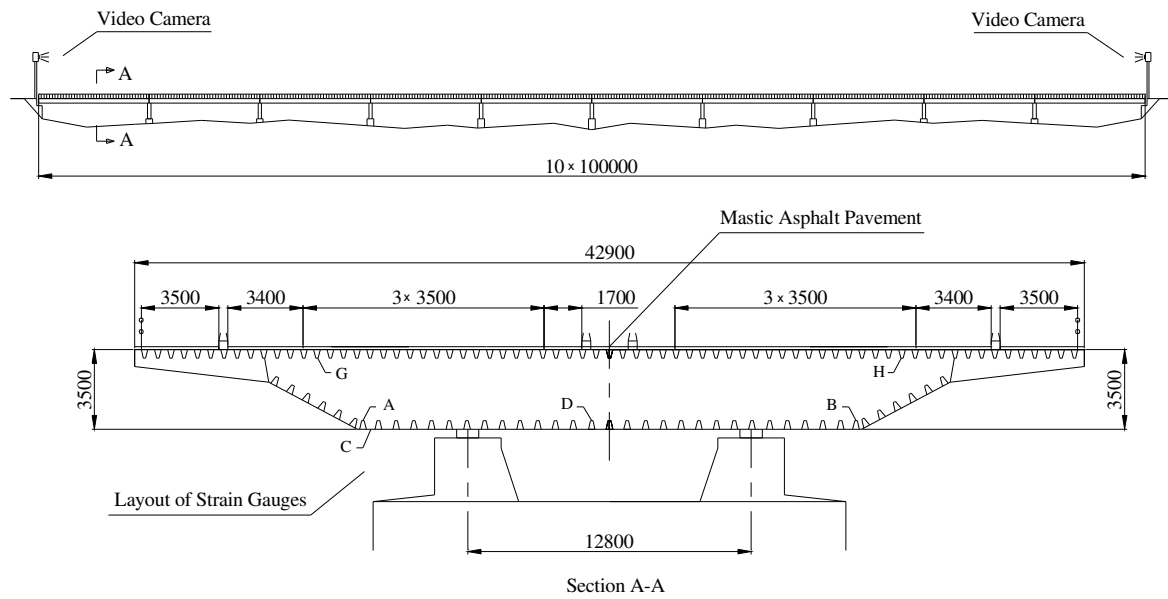


Figure 1. Moerdijk Bridge elevation and cross section with layout of strain gauges (A, B, C, etc.) and video cameras (dimensions in mm).

Strain measurements and video recordings were collected for 11 working days in May-June, 2003. Four strain gauges, A-D in Figure 1, were placed on the bottom longitudinal stiffeners of the steel plate while two (G and H) were positioned on the top stiffeners. All six strain gauges were located at mid-span.

Gross vehicle weights (GVW) are recorded at several WIM stations in the region of the Moerdijk Bridge. For this study, data over a one-month period (28th May – 1st July, 2001) was obtained from the WIM stations closest to the bridge Northbound and Southbound (Figure 2). Histograms for GVW were assembled for each of 13 classes of heavy vehicle, treating northbound and southbound traffic separately. Cars were all considered as being of constant weight, 1500 kg, and length, 4.5m. Buses were assigned a constant weight of 4000 kg and length, 6m, which is justified based on the very small proportion of buses (0.05%) in the total flow.

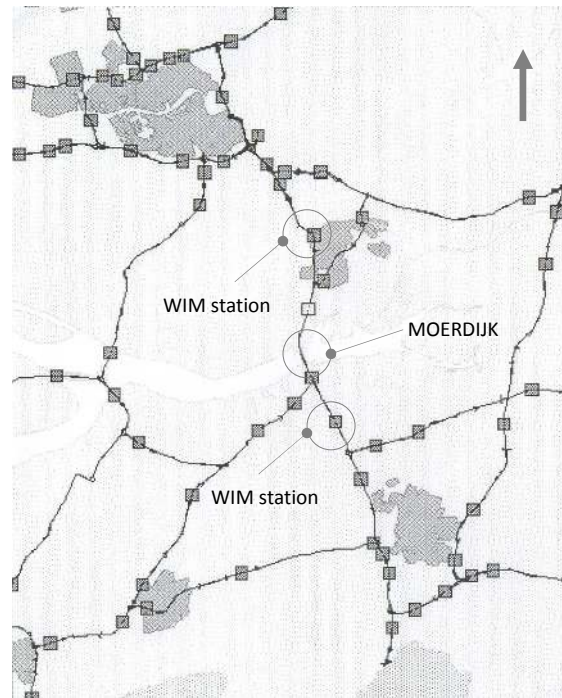


Figure 2. WIM measurement locations in region of Moerdijk Bridge.

In order to determine the influence surface, a three-dimensional Finite Element Model was developed of a complete 100m span using the MSC NASTRAN package (Figure 3). Quadrilateral plate elements were used to represent the top, bottom and side plates, webs and longitudinal trapezoidal stiffeners. In total, there were 47,419 nodes and 59,254 elements in the model.

### 3. Calibration and validation of micro-simulation model

Traffic data, which include observed speed ranges, volumes and vehicle classification information, were used to calibrate and validate the micro-simulation model of the bridge.

The daily traffic volume was subdivided into 10 time periods each of 1-hour duration, from 8:30 to 18:30, for the 11 days of observation. Traffic flow characteristics for each vehicle class varied between the time periods of the micro-simulation model. This resulted in the simulation of free-flowing and congested traffic conditions, depending on the time of day. Figure 4 illustrates actual and simulated traffic scenarios on the bridge. Driver behaviour cannot readily be quantified and is largely based on indirect calibration through the micro-simulation sub-models (car-following gap, gap acceptance and lane changing), which govern the target headway and reaction time of individual vehicles. The procedure is based on model run comparisons to video data sets collected on the bridge and is purely empirical. Numerous experiments under various traffic conditions were conducted in related studies (e.g. Kunzman (1978), Abdulhai (1999), Chu et al. (2003)), in which the driver's target headways and reaction times are

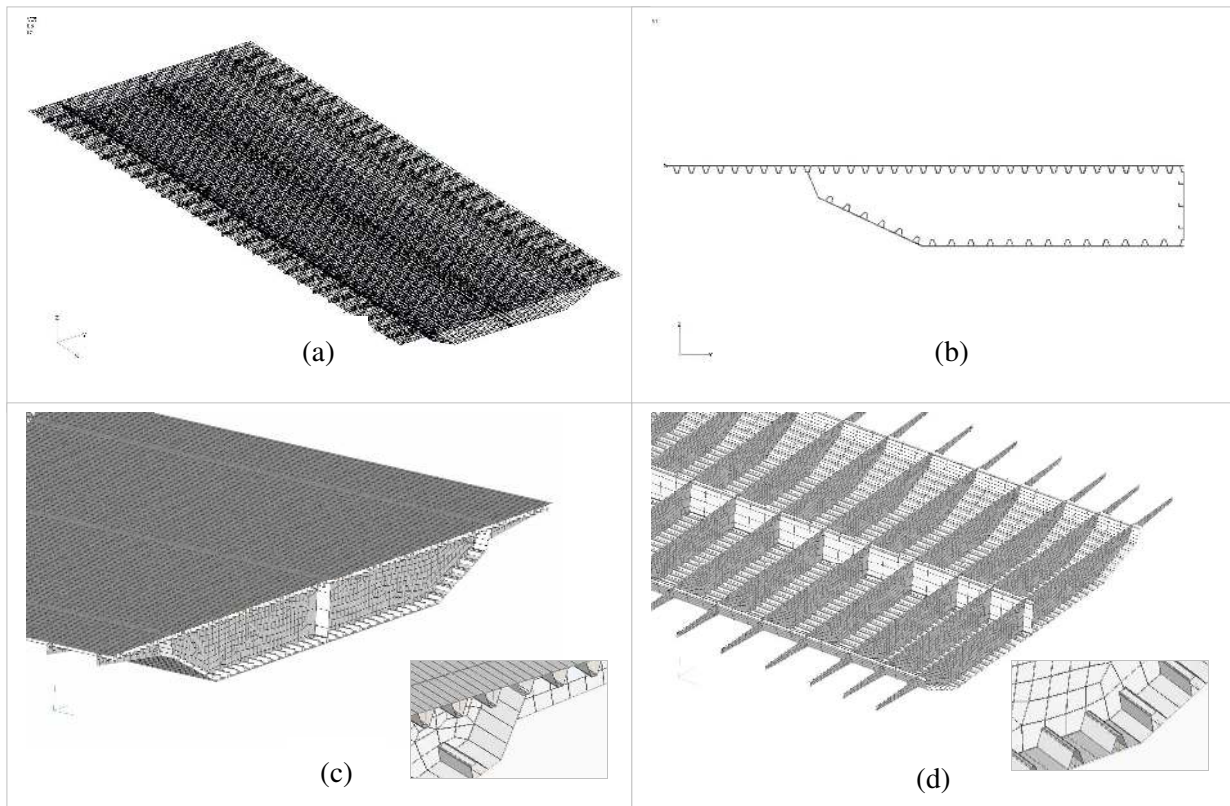


Figure 3. Moerdijk Bridge finite element model: (a) mesh details, (b) half cross-section of the bridge with details of longitudinal stiffeners, (c) rendered solid view, (d) rendered solid view of the bottom plate with details of side plates and transverse diaphragms.

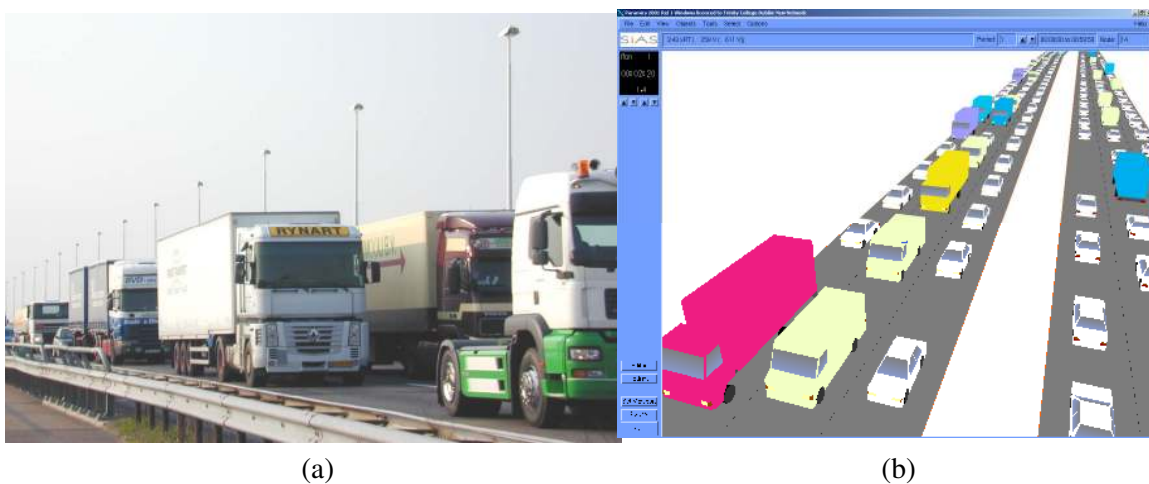


Figure 4. Congested traffic on the Moerdijk Bridge; (a) actual, (b) microsimulation model.

determined to be Normally distributed about their mean values in the ranges of 0.6-2.0 seconds and 0.3-2.0 seconds respectively. In the micro-simulation model used here, the initial driver's target headway and reaction time were assumed to be 1 second (standard deviation = 0.08). These values are based on research of the Transport Research Laboratory (Jeffrey, 1994) and are extracted from observations of freeways in the United Kingdom. In the presence of the field traffic data, to match local conditions for the A16 highway, there was a need to validate the model by calibrating local mean headways and reaction times. The simulation was run with multiple combinations of  $h$  and  $r$ , and for each simulation two key output indicators were computed: average network speed and the maximum vehicle throughput for the 3-lane freeway in each direction. Numerous runs with different combinations of distributions were conducted until the outputs of simulated traffic flow and speed were improved.

Vehicles were manually counted from videotape observations and compared with traffic counts from the micro-simulation model. The lane changing and platoon formation processes in the model were calibrated with observations through the adjustment of the drivers' target speeds, target headway and reaction times. Good correspondence between the model and the observations was achieved by reducing the model target speeds of Southbound and Northbound links to 40 km/h and 17 km/h respectively for the peak hours. Normally distributed drivers' target headway and reaction time distributions were assumed with mean values for both of 0.65 s and standard deviations of 0.48 and 0.45 respectively.

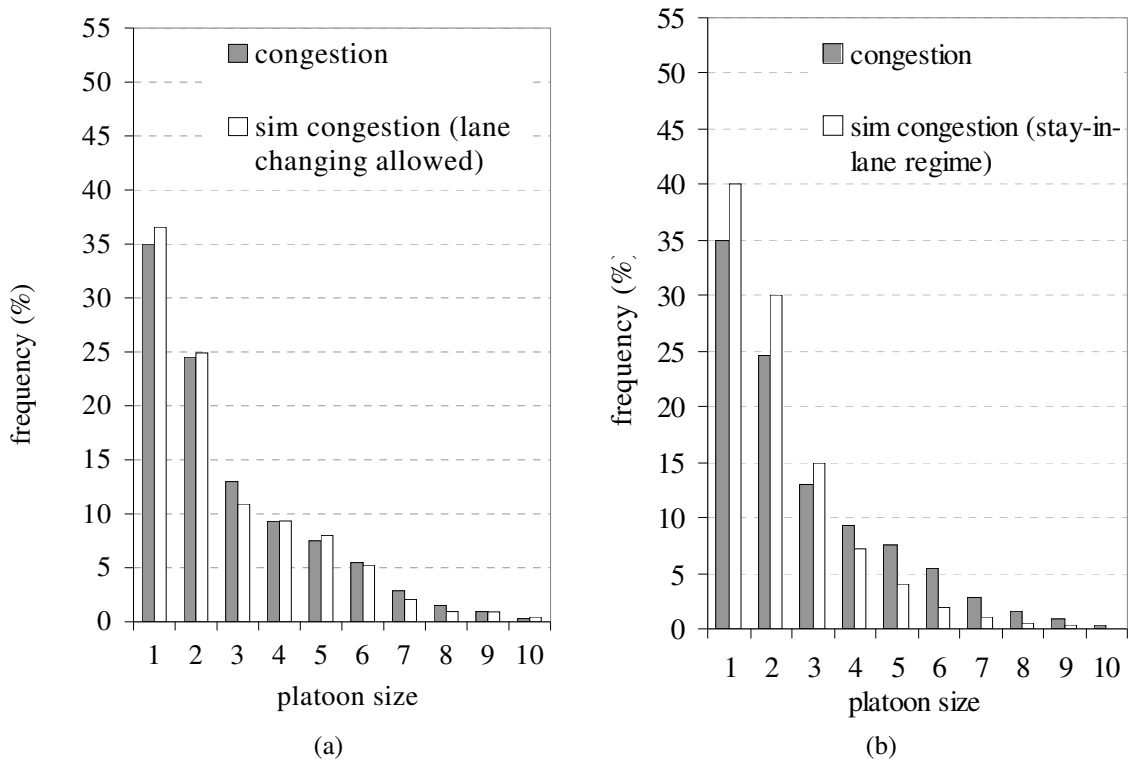


Figure 5. Platoon size distribution: comparison of micro-simulation with field observations. (a) allowing for lane changes, (b) stay-in-lane regime.



Agreement between the resulting platoon frequency distributions in Figure 5(a) demonstrates that realistic car-truck mixes were achieved in the model. To identify the influence of driver behaviour on the formation of truck platoons, the platoon frequency distributions were obtained for the same simulation scenarios of calibrated parameters assuming the “stay-in-lane” regime of traffic, i.e., when lane-changing/overtaking manoeuvres are not allowed between the traffic lanes. Figure 5(b) compares the results. There are a number of large platoons of vehicles in the simulated “stay-in-lane” regime (Figure 5(b)) due to the small gaps between vehicles in the high-density congested phase. However, the frequency of these longer platoons is low. The higher frequency of the longer platoons in the real data is due to faster vehicles being allowed to change lanes.

#### 4. Results

For each vehicle type, the histogram of gross vehicle weight is fitted to a bimodal Gaussian statistical distribution (Castillo 1987, Ang & Tang 1975). Monte Carlo simulation is then used to generate weight data appropriate to the measured histogram. The influence surface, as determined from the FE model for the left- and right-most longitudinal stiffeners, i.e. points A & B (Figure 1) at the bottom plate of the bridge mid-span is then used to determine strain histories from simulated traffic. These points (A & B) represent the most critical mid-span bridge locations under global traffic effects. Local effects (G, H) are only affected by local traffic and are similar to short-span bridges. Hence, they are governed by single heavy trucks or two parallel trucks rather than jams. Simulated vehicle loading patterns with the assigned gross vehicle weights are applied to the bending moment influence surface to calculate its response to traffic loading.

It was not possible to carry out a load test to calibrate or validate the Finite Element model. However, small adjustments were made to the model parameters to achieve a good match between measured and simulated mean maximum-per-hour strains. This calibration process combines the uncertainties in model parameters (such as modulus of elasticity), strain gauge factors and vehicle GVW data. Figure 6 illustrates a typical fragment of averages of four days of peak bending moments from each 10-minute interval. The range and the overall magnitude are similar for measured and simulated moment.

Table 1 gives the mean and standard deviation of the 10-minute peak moments through each of four different days of data. The differences between measured and simulated are within the natural range of variation from day to day. Figure 7 is the histogram of hourly maximum moments over the 11 days and the mean and standard deviations of these hourly maxima. It can be seen that, after calibration, the simulated and measured values match quite well. It is acknowledged that the calibration process can only be considered approximate.

Table 1. Mean and standard deviation of peak moments in 10-minute intervals of four days of data.

	calculated moments, kNm				simulated moments, kNm				average difference
	02-Jun	03-Jun	05-Jun	06-Jun	Day 1	Day 2	Day 3	Day 4	
mean	5168	5749	4878	5158	5318	5555	5870	5363	-5.50%
standard deviation	611	737	797	597	629	645	637	593	-3.09%

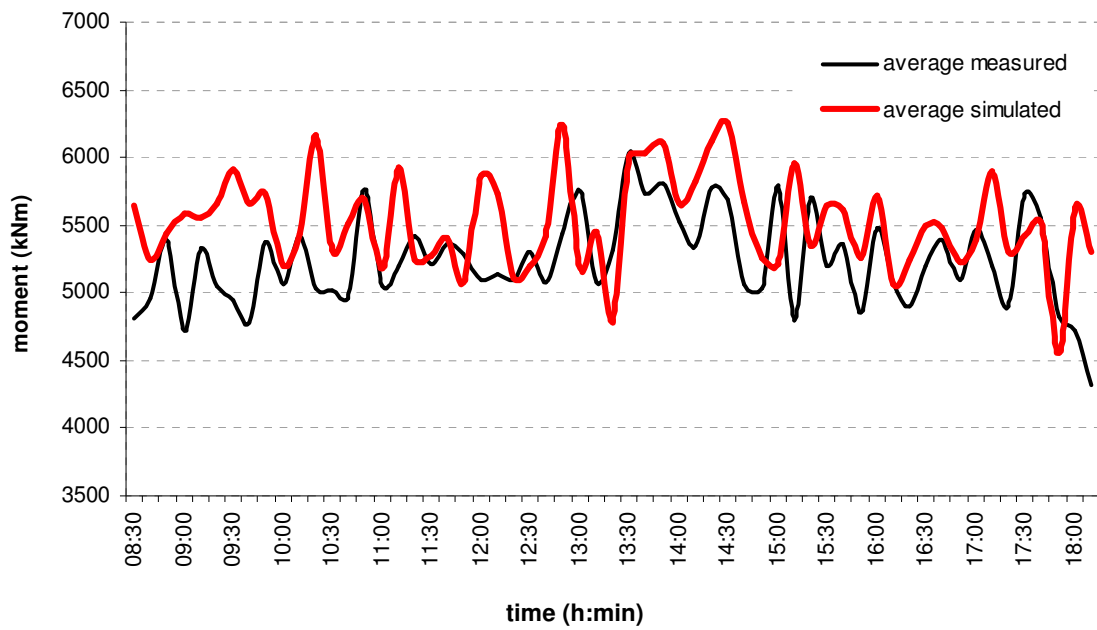
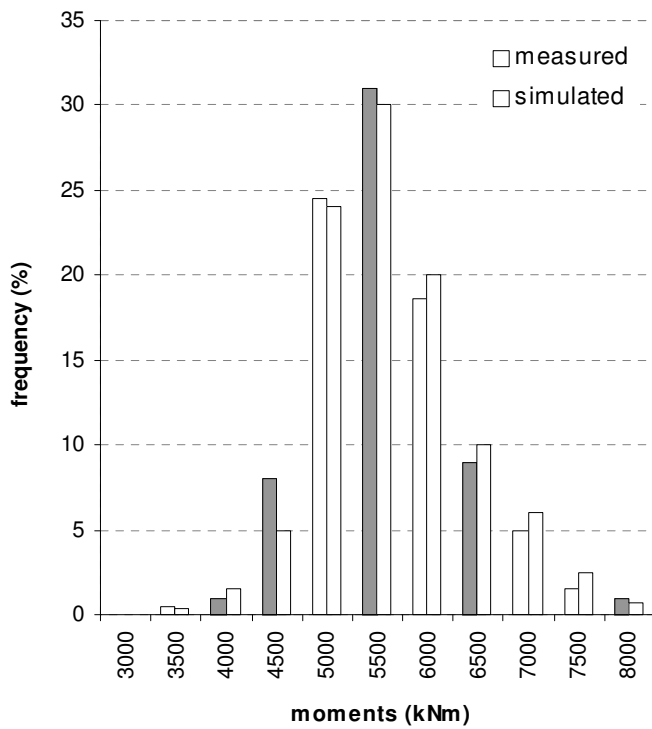


Figure 6. Averages of 4 days of peak moments in 10-minute intervals due to the traffic stream generated from the simulation and those measured on the bridge.



	Bending Moment	
	<i>Measured</i>	<i>Simulated</i>
Mean	5396 kNm	5479 kNm
Standard Deviation	197 kNm	198 kNm

Figure 7. Histograms of hourly maximum bending moment; measured and simulated.

Since in the current work, the bending moment is obtained as the sum of a large number of independent loads coming from the single random vehicle weights, it is to be expected from the central limit theorem that the instantaneous bending moment is approximately normally distributed. Assuming the hourly maximum of the bending moment to be normal, the extreme value *CDF* and *PDF* can be approximated as:

$$F_x(x) = F^n(x) = \Phi^n \left[ \frac{(x - \mu)}{\sigma} \right] \quad (1)$$

$$f_x(x) = nF^{n-1}(x)f(x) \quad (2)$$

for some equivalent number  $n$  of independent Gaussian random variables in the sample, where  $\Phi$  is the standardized normal distribution function and  $\mu, \sigma$  are the mean and standard deviation, respectively, of the Gaussian random variables. Free flowing and congested hours involve different loading scenarios; however, the hours with congested conditions will govern so free flowing traffic is not considered here.

The largest bending moments per hour were selected from the parent population of empirical traffic load effect data observed/simulated between 08:30 and 17:30 for the 11-day period. Based on this selection, sets of 110 hourly moment maxima were derived. Figure 8 illustrates the hourly maximum moment distributions of observed data plotted on Gumbel probability paper but fitted to Equation (2). The maximum likelihood fit to the data corresponded to  $n = 370$  independent samples. However, the likelihood is insensitive to  $n$ . An hour of congested traffic involves a large number of overlapping loading events but it is difficult to determine the number of these that are truly independent. A pragmatic approach is taken here, given the insensitivity of likelihood to  $n$ , and the 'Normal raised to the  $n^{\text{th}}$  power' is simply treated as a distribution that fits the data well.

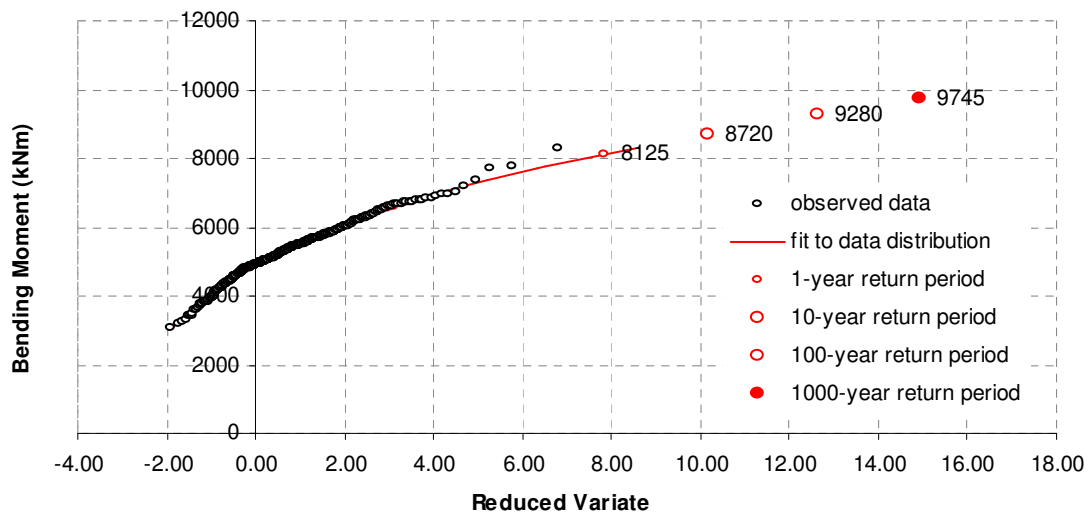


Figure 8. Observed maximum bending moment data on Gumbel Probability Paper.

The estimated extreme distributions have been used to calculate the characteristic (maximum) moment due to traffic, i.e., the expected value of the maximum moment. This is repeated for return periods of 1, 10, 100, and 1000 years. The return value is calculated by assuming that the year has 250 working days, each with 10 hours of heavy traffic. Thus, for example, the 100-year return value is the percentile that corresponds to a probability of exceedance of  $1/(10 \cdot 250 \cdot 100)$  in the 1-hour extreme value distribution. The extrapolated results are summarised in Table 2 where it can be seen that there is a good match between observed and micro-simulated. When vehicles are forced to stay in lane, there is a non-conservative error of 9 to 11%.

Table 2. Characteristic values for bending moment (kNm).

<i>Return Period</i>	<i>Observed</i>	<i>Micro-simulation</i>	<i>Stay-in-Lane</i>	<i>Micro-simulation Error</i>	<i>Stay-in-Lane Error</i>
1 year	8125	8261	7422	1.6%	-8.7%
10 years	8720	8783	7861	0.7%	-9.9%
100 years	9280	9311	8254	0.3%	-11.1%
<b>1000 years</b>	9745	9776	8667	0.3%	-11.1%

The characteristic values are also compared with the corresponding characteristic value for mid-span bending moment calculated in accordance with Eurocode 1, Part 3. The 3D FEM of the bridge was used to compute the moment at location A (Figure 1) where strain measurements were available. Eurocode Load Model 1, consisting of concentrated and uniformly distributed loads, is used to represent the traffic load effect for general verification. Because the carriageway on the bridge deck is divided into two parts, separated by a fixed central barrier, each part, including hard shoulders, is separately divided into five notional lanes (in the Eurocode, every additional 3 m width constitutes a notional lane). Hence, a design moment of 16,604 kNm is found. It should be noted that this value includes an allowance for dynamic amplification which, for a five notional lane bridge, is 1.1. Hence the static value is  $16,604/1.1 = 15,095$  kNm.

The difference is 35.4% between the calculated Eurocode characteristic value and the corresponding observed value (Table 2). This reflects the code's conservatism which is necessary for a general load model for new bridges and valid for a wide range of loading conditions.

## 5. Conclusion

This paper describes the use of micro-simulation techniques to calculate the characteristic loading due to congested traffic on a long-span bridge structure. The driver behaviour and hence the car-truck mix is calibrated through a mixture of traffic parameters and behavioural characteristics. The aggregated output from multiple runs of the calibrated traffic-bridge simulation model was compared to real data collected from the field. Comparisons of micro-simulated and observed /measured variables were performed on hourly and daily bases. The system was calibrated to achieve agreement with the results found for the critical mid-span bending moment of the Moerdijk Bridge. On Gumbel probability paper for maxima, the trend for maximum-per-hour moment followed a very similar trend for micro-simulated and observed data. This shows that the micro-simulation is very effectively reproducing the critical combinations of vehicles (or ones which give the same extreme bending moment). It is evident from the detailed verification analysis of the system outputs that micro-simulation is an excellent

tool for a comprehensive and extensive modelling of bridge traffic loading conditions derived from the real traffic.

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### References

- Abdulhai, B., Sheu, J-B. and Recker, W., 1999. *Simulation of ITS on the Irvine FOT area using Paramics 1.5 scalable microscopic simulator – phase 1: model calibration and validation*. California PATH Research Report UCB-ITS-PRR-99-12. University of California at Irvine.
- Ang, A. H-S. and Tang, W.H., 1975. *Probability concepts in engineering planning and design*. New Year: John Wiley.
- Bruls, A., Croce, P., Sanpaolesi, L. and Sedlacek, G., 1996. EVN1991-Part 3: Traffic loads on bridges; Calibration of load models for road bridges, *Proceedings of IABSE Colloquium*, Delft: IABSE-AIPCIVBH, 439-453.
- Buckland, P.G., McBryde, J.P. and Francis, P.D., 1978. Traffic loading of long span bridges. *In: Conference on bridge engineering proceedings*, Transportation Record Research, 146-154.
- Buckland, P.G., 1981. Recommended design loads for bridges (Committee on loads and forces on bridges of the committee on bridges of the structural division), *Journal of the Structural Division*, ASCE, 1161 – 213.
- Castillo, E., 1987. *Extreme value theory in engineering*. London: Academic Press.
- Chu, L., Yang, X., 2003. Optimisation of the ALINEA ramp-metering control using generic algorithm with micro-simulation. Transportation Research Board 82<sup>nd</sup> Annual Meeting, Preprint CD-ROM, Washington DC, January 12-16, 2003.
- Ditlevsen, O. and Madsen, H., 1997. Stochastic vehicle-queue-load model for large bridges. *Journal of Engineering Mechanics*, 120 (9), 1829-1847.
- European Committee for Standardisation. EVN 1991-2:2003, EC1: 2003. *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges*.
- Flint and Neill Partnership for TRRL., 1986. *Interim design standard: long span bridge loading*. Transport and Road Research Laboratory: Crowthorne (Contractor Report 16).
- Fritzsche, H.T., 1994. A model for traffic simulation. *Traffic Engineering and Control*, 35 (5), 317 – 321.
- Jeffreys, W., 1994. *Motorway lane discipline: motorway traffic modelling study*. Technical Report, Transportation research Laboratory: United Kingdom, March 1994.
- Kunzman, W., 1978. Another look at signalised intersection capacity. *ITE Journal*, 48 (1), 12-15.
- Lieberman, E. and Rathi, A.K., 1999. *Traffic simulation*. New York.
- Nowak, A.S., 1991. Bridge load models. *In: R.Heywood, Austroads bridges conference*, 13-15 November 1991 Brisbane, 27-44.
- Nowak, A.S., 1999. *Calibration of LRFD bridge design code*. Transportation Research Board: National Cooperative Highway Research Program, Report 368.
- Rafael, P., Saldana, Winfer, C. Tabares., 2000. Traffic modeling on high performance computing systems. *In: Proceedings of the philippine computing science congress (PCSC 2000)*. Computing Science of the Philippines, 2000.
- Wiedemann, R., 1974. *Simulation des Straßenverkehrsflusses*. Schriftenreihe des Instituts für Verkehrswesen der Universität Karlsruhe, Vol 8.