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# A comparative study of a bridge traffic load effect using micro-simulation and Eurocode load models

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**ABSTRACT:** Micro-simulation, a process by which individual vehicles and driver behaviour are simulated, is used in this study to assess the effects of congested traffic load on bridges. The Eurocode for traffic loading on long-span bridges was derived from simulations of convoys of trucks at small bumper-to-bumper distances. Real congested traffic includes a mix of cars and trucks with a variety of congestion patterns, which reduce the load effects. Micro-simulation is used in this study to generate more realistic traffic loading scenarios, based on real traffic data from a Polish site. The results are then extrapolated to determine characteristic maximum effects for two return periods. The Eurocode is shown to be conservative, and micro-simulation based assessment is therefore valuable for problem long-span bridges.

## 1 INTRODUCTION

### 1.1 *Motivation*

It is generally accepted (O'Connor & OBrien 2005) that short span bridges are governed by small numbers of vehicles in free flowing traffic, with an allowance for dynamic amplification. Long span bridges on the other hand are governed by congested conditions. In congestion, a greater number of vehicles are present, at much closer spacing, but no allowance for dynamics is appropriate.

Since April 2010, the Eurocodes are the legal standards for structural design throughout the European Union. Eurocode 1 - Part 2 (2003) deals with bridge loading, but its provisions apply only to the design of new bridges with spans up to 200 m. No provisions are made regarding the traffic loading for bridge safety assessment.

In the absence of code provisions, common design practice for traffic loading on long-span bridges usually relies on conservative assumptions about the traffic and does not consider the variability of congestion patterns and driver behaviour. This is important for the safety assessment of existing bridges as it could make the difference between performing repair/rehabilitation or not. Maintenance operations for long-span bridges are expensive and therefore such conservative assumptions may play a decisive role, resulting in unnecessary expenditure and traffic disruption.

For longer spans, Eurocode 1 (2003) was calibrated assuming that congested conditions govern (Flint & Jacob 1996, Prat 2001). Small axle-to-axle distances were assumed in truck-only scenarios (Prat 2001). It is reasonable to assume that this is conservative as truck-only situations and complete gridlock are both rare cases in practice.

In this paper, micro-simulation is used to generate traffic congestion based on truck weight data collected from the A4/E40 near Wroclaw in Poland. The load effects on three sample bridges are calculated for 1000 hours of congestion, deemed to represent 1 year of traffic (250 working days and 4 congested hours per day). The results are then extrapolated to 75-year (for bridge assessment) and 1000-year return periods (for new bridge design, to compare with Eurocode 1).

### 1.2 *Load models for long-span bridges*

The available models for long-span bridge traffic loading take into account the variability of truck weights, but they typically assume full stop traffic with a mix of cars and heavy vehicles at minimum bumper-to-bumper distances (Ivy et al. 1954, Buckland 1981, Flint & Neill Partnership 1986, Ditlevsen & Madsen 1994, Nowak & Lutomirska 2010). The minimum inter-vehicle distance is often assumed, rather than based on real data.

It is generally accepted that truck weights from a static weigh station are biased as overloaded trucks will seek to avoid being weighed. Fortunately, weigh-in-motion (WIM) systems are now commonplace and large databases of data are available. WIM systems provide data on gross and axle weights, axle spacings, (axle-to-axle) gaps between vehicles and headways. This data can be supplemented by simpler and less expensive counting stations which often use magnetic induction loops to gather information on numbers and classification of vehicles. The majority of operating WIM and induction loop installations only work reliably at high speed. The result is that there is a dearth of information for congested traffic.

### 1.3 Traffic micro-simulation

Traffic micro-simulation (i.e., simulating the motion of individual vehicles) is able to replicate the interaction between vehicles, effectively generating different congestion patterns. Significantly, traffic measured during free-flowing conditions can be used as input to a micro-simulation model to generate corresponding congested traffic.

OBrien et al. (2010) were the first to use micro-simulation in a study of bridge traffic loading. They used commercially available micro-simulation software to model driver behaviour in congested conditions on a 100 m span bridge in the Netherlands. They found that lane changing resulted in convoys of trucks that increased characteristic load effect for the bridge considered by about 10%.

Cellular automata, a similar micro-simulation approach (Nagel & Schreckenberg 1992), has also recently been used for bridge traffic loading by dividing the bridge into 7.5 m cells (Chen & Wu 2011). However, this does not allow for the variability in vehicle lengths and inter-vehicle gaps.

In this paper, the car-following Intelligent Driver Model is used (Treiber et al. 2000a). The flow of two classes of vehicle (cars and trucks) running on a single-lane road is studied. The simulations are carried out using an in-house program called *Simba* (*Simulation for Bridge Assessment*).

## 2 THE EUROCODE LOAD MODEL

The Eurocode load models are based on one-week traffic data collected on the A6 motorway near Auxerre (France), deemed to be representative of European traffic (Prat 2001). The Load Model 1 for general and local verifications of bridges up to 200 m consists of:

- a characteristic uniformly distributed load (UDL)  $q_{ik}$  and;

- a characteristic tandem system (TS) load  $Q_{ik}$ , where the two axles are spaced 1.20 m apart (Figure 1).

Their values depend on the notional lane number  $i$ . The notional lane is 3 m wide. Therefore, there are often more loaded lanes than actual lanes.

For the first lane  $i = 1$ , the values are  $q_{1k} = 9$  kN/m<sup>2</sup> and  $Q_{1k} = 300$  kN. For other lanes, the characteristic values reduce, reaching a minimum of  $q_{ik} = 2.5$  kN/m<sup>2</sup> and  $Q_{ik} = 0$  kN in the fourth and subsequent lanes (Table 1). The characteristic values correspond approximately to a 1000-year return period (5% probability of exceedance in 50 years).

Adjustment factors,  $\alpha_{Qi}$  and  $\alpha_{qi}$  are decided at national or network level and depend on the traffic features, if such information is known. Their default values are unity. The methodology implemented in this paper may be used to quantify the adjustment factors for a given bridge or group of bridges.

For shorter spans, the Eurocode load model includes an allowance for dynamic amplification. However, since congestion governs for spans in excess of about 50 m (Flint & Jacob 1996), no dynamic amplification has been allowed for in this study (Bruls et al. 1996, Prat 2001).

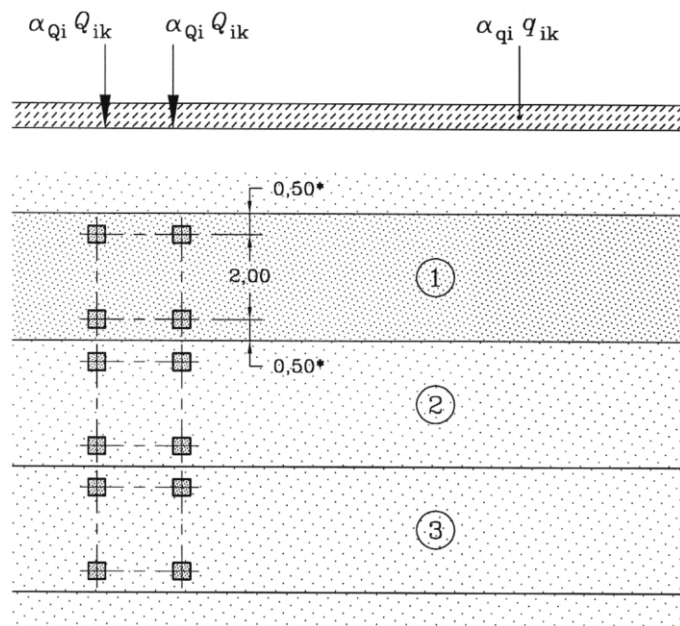


Figure 1. Load model 1 (taken from Eurocode 1, 2003)

Table 1. Characteristic load values (adapted from Eurocode 1, 2003). TS = Tandem System; UDL = Uniformly Distributed Load.

Lane number	TS $Q_{ik}$ (kN)	UDL $q_{ik}$ (kN/m <sup>2</sup> )
1	300	9
2	200	2.5
3	100	2.5
4 or more	0	2.5

### 3 MICRO-SIMULATION

#### 3.1 Introduction

Micro-simulation is widely used today in traffic studies. Models vary in their levels of complexity and accuracy. Car following models consider driver behaviour within a single lane, simulating the interaction between a vehicle and its leader (Brackstone & McDonald 1999, Orosz et al. 2010). More complex lane-changing models are also available for multi-lane traffic and are better suited to modelling the formation of truck convoys as traffic becomes congested.

Macro-simulation considers only the aggregate properties of a traffic stream whilst micro-simulation considers the motion of individual vehicles. Aggregate properties can be extracted from micro-simulation models, but this is insufficient to validate all results. Unfortunately, microscopic (individual vehicle) data is difficult to find with the result that models are often only calibrated at an aggregate level (Hidas & Wagner 2004).

Suitable data for calibrating lane-changing models is even more difficult to collect, since it requires, for example, video tracking over a long stretch of road to capture the interaction between all the vehicles involved in lane-changing manoeuvres. In fact, along with macroscopic models, some car-following models are used to describe the global effects of multi-lane traffic (Treiber et al 2000b, Schönhof & Helbing 2007).

#### 3.2 The IDM car-following model

Treiber et al. (2000a) present the Intelligent Driver Model (IDM), a car-following model that gives a good match, at macroscopic level, with real congested traffic (Treiber et al. 2000b, Helbing et al. 2009). It has a modest number of physically-meaningful parameters and has been calibrated with real trajectory data (Kesting & Treiber 2008, Hoogendoorn & Hoogendoorn 2010, Chen et al. 2010). Results from the calibrated Intelligent Driver Model are comparable with more complex models (Brockfeld et al. 2004, Punzo & Simonelli 2005).

The Intelligent Driver Model has been implemented for this study in the program *Simba*. Central to this approach is an acceleration function:

$$\frac{dv}{dt} = a \left[ 1 - \left( \frac{v(t)}{v_0} \right)^4 - \left( \frac{s^*(t)}{s(t)} \right)^2 \right] \quad (1)$$

where  $a$  is the maximum acceleration;  $v_0$ , the desired speed  $v(t)$  is the current speed,  $s(t)$  is the current gap to the vehicle in front and  $s^*(t)$  is the minimum desired gap, given by:

$$s^*(t) = s_0 + Tv(t) + \frac{v(t)\Delta v(t)}{2\sqrt{ab}} \quad (2)$$

In Equation (2), the term  $s_0$  is the minimum bumper-to-bumper distance,  $T$  is the safe time headway,  $\Delta v(t)$  is the velocity difference between the current vehicle and the vehicle in front and  $b$  is the comfortable deceleration. An advantage of this approach is that just five measurable parameters are sufficient to capture driver behaviour.

The IDM can be extended to multi-lane simulation with the lane-changing model MOBIL (Kesting et al. 2007). However, this work is restricted to single-lane simulations.

### 4 CONGESTED TRAFFIC STATES

#### 4.1 Inducing congestion

To generate congestion, Treiber et al. (2000a, b) have applied flow-conserving inhomogeneities. This is achieved through a local variation of the parameters. For example, the desired speed,  $v_0$ , is decreased or the safe time headway,  $T$ , is increased. These changes have a similar effect as an on-ramp bottleneck. This approach has been successfully applied to *single-lane* simulations for simulating congested traffic on some *multi-lane* German motorways (Treiber et al. 2000b).

In this paper, the inhomogeneity is generated by increasing the safe time headway,  $T$ , to a new level,  $T'$ , downstream of where the congestion is sought. Treiber et al. (2000b) suggest that this is more effective than decreasing  $v_0$ .

A study of the different possible congestion patterns on long-span bridges and their load effects for a single lane and identical vehicles has been presented by Lipari et al. (2010).

#### 4.2 Bottleneck strength

Bottleneck strength,  $\Delta Q$ , is a measure of the strength of the congestion-inducing phenomenon. It is defined as the difference between the outflow,  $Q_{\text{out}}$ , with the original parameter set and the outflow,  $Q'_{\text{out}}$ , with the modified set, in this case with the modified safe time headway  $T'$ :

$$\Delta Q(T') = Q_{\text{out}}(T) - Q'_{\text{out}}(T') \quad (3)$$

Treiber et al. (2000b) refer to  $Q_{\text{out}}$  as the *dynamic capacity*, i.e., the outflow from a congested state. It is less than the static capacity  $Q_{\text{max}}$ , which is only achieved in free equilibrium traffic. For the parameter set chosen, the dynamic capacity is quite close to the static capacity,  $Q_{\text{max}}$ .

### 4.3 Congested traffic states

Table 2 lists six forms of congestion that can occur (Treiber et al. 2000b, Schönhof & Helbing 2007). The traffic can take up any of these states, depending on the combination of inflow  $Q_{in}$  and bottleneck strength  $\Delta Q$ . Combinations of these congested states are also possible and previous traffic history can influence the state of congestion.

Table 2 – Definitions of various traffic states

Acronym	Explanation of traffic state
FT	Free traffic
MLC	Moving localized cluster
PLC	Pinned localized cluster
SGW	Stop and go waves
OCT	Oscillatory congested traffic
HCT	Homogeneous congested traffic

## 5 MODEL AND SIMULATION PARAMETERS

### 5.1 Traffic stream

For this study, the input vehicle stream is made up of traffic recorded over 91 days on the transcontinental A4/E40 near Wroclaw (Poland). The original flow is manipulated in order to increase the inflow  $Q_{in}$  up to 1500 veh/h, while keeping the original overall truck and car proportions.

The inflow rises from 0 up to  $Q_{in}$  in the first 30 minutes, then it is kept constant for the next two hours. After that, there is no inflow for the next 4 hours, in order to allow the traffic queue discharge (Figure 2). It is assumed that such a congestion event occurs once per working day. In order to obtain 1 year of traffic, 250 such congestion events are simulated, under the assumption of 250 working days per year.

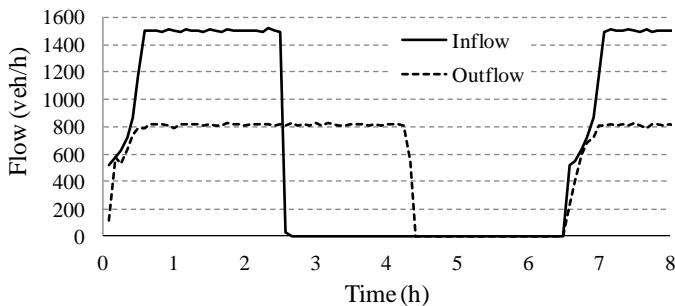


Figure 2. Example of simulated inflow and outflow from congestion (aggregation time 5 min).

The traffic stream is classified into two vehicle classes: cars and trucks. A car is defined as a vehicle weighing less than 3.5 tonnes. All the other vehicles are classified as trucks. A total of 842 500 vehicles

are injected of which 21.1% are, by this definition, trucks. Only 24% of the trucks have four or more axles. The histogram of truck gross vehicle weights is shown in Figures 2 and 3. There is a high proportion of light trucks weighing less than 50 kN. However, magnification of the overloaded trucks (over 44 t) shows there are trucks weighing up to 737 kN (Figure 3).

Each vehicle is assigned a parameter set to define the driver behaviour (Table 3). The parameters are based on those used by Treiber et al. (2000b). Trucks are assigned a smaller desired speed than cars, reflecting their lower speed limit. All the parameters are constant throughout the vehicle population.

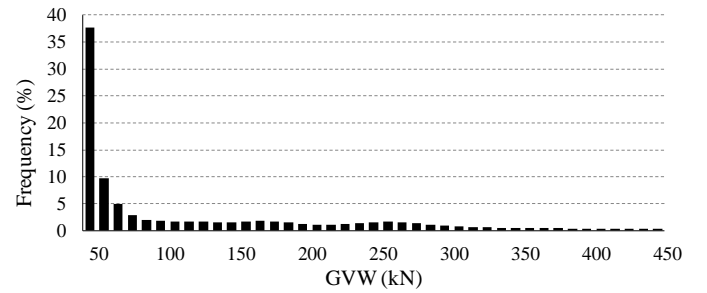


Figure 2. Truck GVW histogram (GVW < 450 kN).

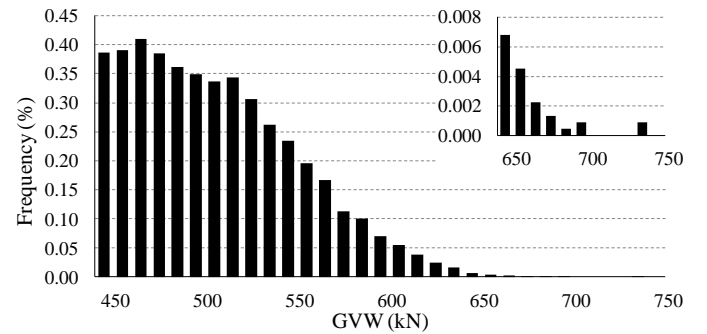


Figure 3. Truck GVW histogram (GVW > 440 kN).

Table 3 - Model parameters of IDM and MOBIL

	Cars	Trucks
Desired velocity, $v_0$	120 km/h	80 km/h
Safe time headway, $T$	1.6 s	1.6 s
Maximum acceleration, $a$	0.73 m/s <sup>2</sup>	0.73 m/s <sup>2</sup>
Comfortable deceleration, $b$	1.67 m/s <sup>2</sup>	1.67 m/s <sup>2</sup>
Minimum jam distance, $s_0$	2 m	2 m

### 5.2 Road capacity estimation

It is not straightforward to calculate the static and dynamic traffic flow capacity  $Q_{max}$  and  $Q_{out}$  of a road for real traffic (Treiber et al 2000b, Treiber & Kesting 2009). Treiber et al. (2000b) calculate the capacities  $Q_{max}$  and  $Q_{out}$  for single-lane simulation with the car parameter set shown in Table 1. How-

ever, the truck presence decreases the road capacity due to the non-homogenous driver population.

In order to compute the reduced dynamic capacity  $Q'_{out}$ , it is assumed that the same reduction factor applies as for the (static) capacity  $Q_{max}$ . In fact, the capacity  $Q_{max}$  can be estimated theoretically, by imposing equilibrium traffic and maximizing the flow condition. The equilibrium traffic condition implies  $dv/dt = 0$  in Equation (1) and  $\Delta v = 0$  in Equation (2). Equation (2) is substituted into Equation (1) and solved for the gap,  $s$ . Hence the equilibrium gap  $s_e$  for a particular speed  $v$  is:

$$s_e = (s_0 + vT) \left[ 1 - \left( \frac{v}{v_0} \right)^4 \right]^{-\frac{1}{2}} \quad (3)$$

Note that there is no longer a dependence on time. Then, the equilibrium flow  $Q_e$  is numerically maximized using the well-known fundamental equation of traffic (which links flow  $Q$  and density  $k$  by means of the speed  $v$ ) and the so-called “micro-macro link” (which links density  $k$  and gaps  $s$ , (Treiber et al. 2000b)):

$$Q_e(v) = kv = \frac{v}{s_e + l} \quad (3)$$

where  $l$  is the length of the vehicles.

Doing so, the maximum theoretical flow  $Q_{max}$  for identical vehicles with the parameters shown in Table 3 (and assuming 5 m length) is 1743 veh/h. The introduction of 21% of longer vehicles (assumed to be 12 m long), decreases  $Q'_{max}$  down to 1680 veh/h (-3.6%). Finally, the dynamic capacity  $Q_{out}$  is scaled by the same factor, thus obtaining  $Q'_{out} = 1628$  veh/h.

### 5.3 Road geometry and bottleneck strength

A single-lane 5000 m long road is considered. No on- or off-ramps are included. The safe time headway is  $T$ , from 0 to 3400 m (see Table 2), then increases linearly to 4000 m until it reaches the value  $T' = 6.4$  s. As illustrated in Figure 2, this procedure creates about 4 hours of congestion. The average outflow from congestion  $Q'_{out}$  is 798 veh/h, so  $\Delta Q = 830$  veh/h, according to Equation (3).

This bottleneck strength value can be seen as being equivalent to the injection of an on-ramp flow equal to  $\Delta Q$  into the main traffic, which is considered here as a typical recurrent heavy congestion scenario. A lower value is likely to return smaller load effects (Lipari et al. 2010), whereas greater values are likely to be too strong to be realistic, suggesting, for instance, a road design fault (Schönhof & Helbing 2007). Ideally, the congestion frequency of the site under consideration should be known, but this information is rarely available.

### 5.4 Congestion pattern

Spatio-temporal speed plots are useful to visualize congested patterns. A virtual detector is placed every 500 m and aggregates the individual speeds over 1 min, similarly to actual traffic sensors. In order to better identify congestion, the speed axis is plotted upside down (Figure 4).

The resulting congested pattern is a combination of HCT (Table 2) near the inhomogeneity and OCT behind. The average speed around the HCT area is 7.5 km/h.

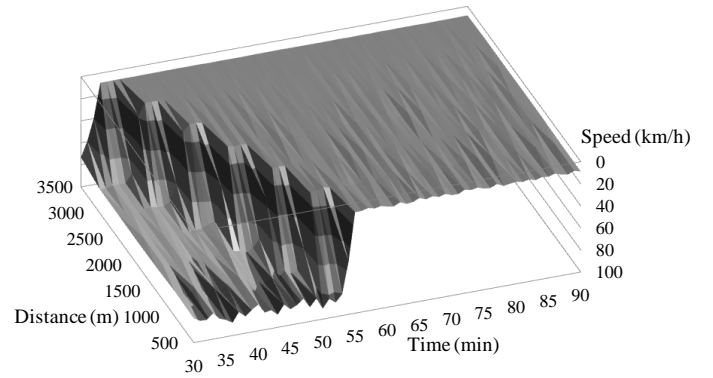


Figure 4. Spatio-temporal speed plot for typical congestion.

### 5.5 Sample bridges

In the following, three simply-supported continuous beams are considered:

- a 50-m long single-span;
- a 100-m long bridge made up of two 50 m spans;
- a 200-m long bridge made up of three spans, respectively 58, 84 and 58 m long.

The load effects considered are sagging moment in the single-span (Load effect 1), and hogging moments at the central support for the multi-span bridges (Load effects 2 and 3), illustrated in Figures 5-7. All of the bridges are centred at 3000 m on the road, in the area preceding the inhomogeneity, and are therefore affected by the HCT state.

## 6 EUROCODE-BASED DESIGN

The Eurocode load model LM1 is placed in the worst position for the relevant load effect, according to the influence line theory for the selected load effects. A lane width of 3 m is assumed, so the UDL applied is 27 kN/m.

The shape of the influence lines and the location of the imposed traffic loading are depicted in Figures 5, 6 and 7. The characteristic design values are shown in Table 4. Notably, load effect 3 (200 m long bridge) is not included among the influence lines for the Eurocode calibration (Prat, 2001).

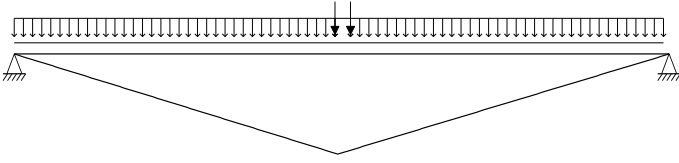


Figure 5. Influence line and load position for mid-span bending moment (Load effect 1 – 50 m bridge).

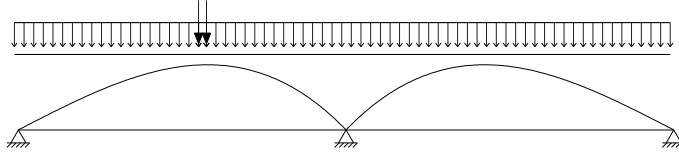


Figure 6. Influence line and load position for hogging moment at central support (Load effect 2 – 100 m bridge).

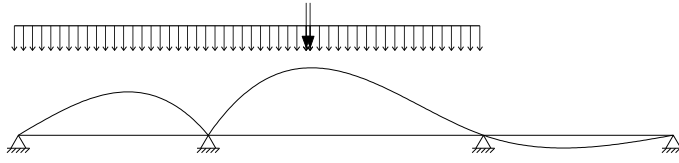


Figure 7. Influence line and load position for hogging moment at central support (Load effect 3 – 200 m bridge).

Table 4 - Eurocode-based design values

Bridge	Spans (m)	Load effect	Characteristic value (kNm)
50 m	50	1 Sagging moment	15 758
100 m	50 + 50	2 Hogging moment	11 322
200 m	58 + 84 + 58	3 Hogging moment	20 616

## 7 MICROSIMULATION-BASED DESIGN

### 7.1 Introduction

In order to compute the 1000-year return period required in the Eurocode 1 (2003) and a 75-year return period for bridge assessment, it is necessary to extrapolate the generated traffic data. The maximum load effect per each congestion event (or day) is computed.

Table 5 gives some basic statistical results for the maximum per day recorded load effects.

Table 5 - Statistics from the simulation (values in kNm)

Bridge	Mean	Standard deviation	Max
50 m	7134	784	9451
100 m	5258	677	7370
200 m	8369	1131	11 606

### 7.2 Estimation

After filtering some outlying low values, the daily maximum data is fitted using the Generalized Extreme Value (GEV) distribution, given by:

$$G(s) = \exp \left\{ - \left[ 1 - \xi \left( \frac{s - \mu}{\sigma} \right) \right]_+^{1/\xi} \right\} \quad (3)$$

where  $[h]_+ = \max(h, 0)$  and  $\mu$ ,  $\sigma$ ,  $\xi$ , are the location, scale and shape parameters respectively. The probability density function (PDF) is:

$$g(s; \theta) = G(s; \theta) \cdot \sigma^{-1} \left\{ 1 - \xi \left( \frac{s - \mu}{\sigma} \right) \right\}^{1/\xi - 1} \quad (3)$$

Maximum likelihood estimation is used. The parameter set that is most likely to yield the  $n$  observed data points,  $y_i$ , is determined by maximizing the likelihood function:

$$L_y(\theta; y) = \prod_{i=1}^n g(y_i; \theta) \quad (3)$$

Maximization of the log-likelihood function is equivalent, since the logarithm is a monotonic function. Hence, maximization is carried out using:

$$\begin{aligned} \log [L(\theta; y)] &= \sum_{i=1}^n \log [g(y_i; \theta)] \\ &= -n \log \sigma - (1 - 1/\xi) \sum_{i=1}^n \log s_i \\ &\quad - \sum_{i=1}^n s_i^{1/\xi} \end{aligned} \quad (3)$$

Where:

$$s_i = 1 - \xi \left( \frac{y_i - \mu}{\sigma} \right) \quad (3)$$

See Coles (2001) for further details on the estimation procedure used.

### 7.3 Statistical extrapolation

The probability of the load effect values  $z_{1000}$  and  $z_{75}$  corresponding to 1000-year return period (250 000 working days) and 75-year return period (18 750 working days) are:

$$P(z_{1000}) = 1 - \frac{1}{T(z_{1000})} = 0.999996 \quad (3)$$

$$P(z_{75}) = 1 - \frac{1}{T(z_{75})} = 0.999947 \quad (3)$$

The corresponding standard extremal variates (for use with Gumbel probability paper) are:

$$SEV_{1000} = -\log [-\log (0.999996)] = 12.43 \quad (3)$$

$$SEV_{75} = -\log [-\log (0.999947)] = 9.85 \quad (3)$$

With the parameters of the distributions estimated, the extrapolated load effects are determined using Equations (11) and (12) and the inverse of Equation (6). The results are given, along with the distribution parameter estimates in Table 6. An example extrapolation on Gumbel probability paper is given in Figure 8.

Table 6. Parameters of the GEV distributions and estimated 1000-year and 75-year characteristic values (kNm)

Bridge	$\mu$	$\sigma$	$\zeta$	$z_{1000}$	$z_{75}$
50 m	6842	624.9	-0.075	11 896	11 192
100 m	9124	482.7	-0.068	9124	8538
200 m	8163	780.7	-0.071	14 620	13 698

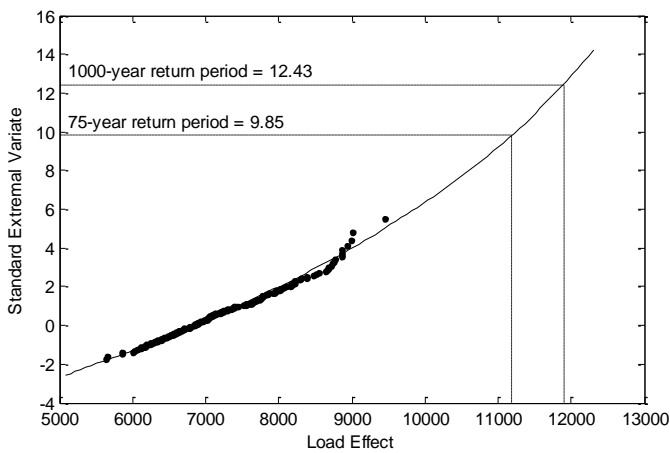


Figure 8. Example extrapolation on Gumbel probability paper for load effect 1.

## 8 COMPARISON

To compare the results of the microsimulation to the Eurocode values, ratios are used, similar to the Notional Load Model Ratio (NLMR) used by O'Brien et al. (2005). The NLMR is the characteristic load effect calculated from the simulations (Table 6), divided by the corresponding value found using the notional model (Eurocode) (Table 4). These values are given in Table 7 and plotted in Figure 9.

Table 7. Notional Load Model Ratio for 1000-year and 75-year load effect

Bridge	Load effect	NLMR <sub>1000</sub>	NLMR <sub>75</sub>
50 m	1 Sagging moment	0.75	0.71
100 m	2 Hogging moment	0.81	0.75
200 m	3 Hogging moment	0.71	0.66

Although heavy congestion is applied for 4 hours per working day, it can be seen that the extrapolation leads to values 29 to 19% lower than the Eurocode's for the design of new bridges. The value for the safe-

ty assessment of existing bridges allows a further reduction by roughly 5%. Probably, a site-specific analysis of the congestion frequency could allow for further reductions.

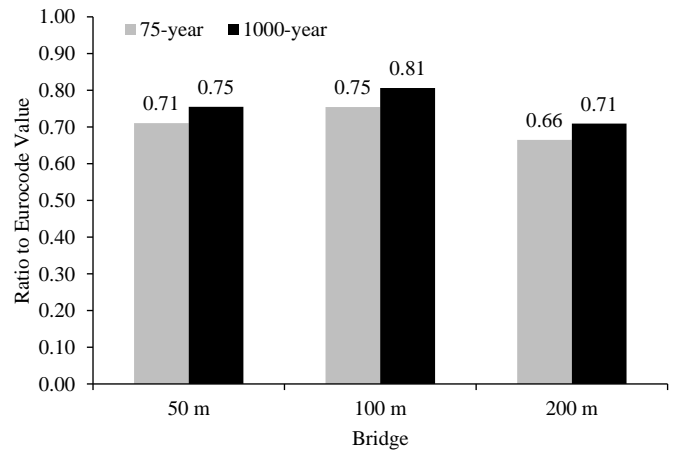


Figure 9. Ratio of microsimulation to Eurocode values for 75- and 1000-year return periods.

## 9 SUMMARY

This paper investigates the load effect from real traffic weight measurements on three sample long-span bridges. Single-lane traffic runs are simulated across the bridges using an in-house micro-simulation tool.

The Eurocode load model, as well as other load models, neglects different congestion patterns, effectively assuming a standstill queue. The car-following model used here is able to reproduce observed congestion patterns.

A total of 1000 hours of heavy congestion is simulated, deemed to represent 1 year of traffic on a busy single-lane road, with an inflow at about 90% of the road capacity. The simulation outputs are extrapolated to 75 years, which is a reasonable return period for bridge assessment. The results are also extrapolated to 1000 years, in order to draw a comparison with Eurocode 1.

It is found that the extrapolated values are about 25% less than the Eurocode provisions for the design of new bridges, whereas the values for bridge assessment allow for a further 5% reduction.

The methodology reported here may be used to adjust the Eurocode provisions when site-specific data is available for a single-lane road.

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