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Authors(s)	Caprani, Colin C., Lipari, Alessandro, O'Brien, Eugene J.
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Load effect of multi-lane traffic simulations on long-span bridges

C.C. Caprani

Dublin Institute of Technology, Dublin, Ireland

A. Lipari & E.J. OBrien

University College Dublin, Dublin, Ireland

ABSTRACT: The traffic loading of long-span bridges is governed by congestion. Real-world observations show that congestion can take up different forms. Nevertheless, most previous studies on bridge traffic loading considered only a queue of standstill vehicles. In this paper, a micro-simulation tool is used for generating congested traffic on a two-lane same-way roadway. The total load is computed for a sample long-span bridge. Different congestion patterns are found and they are studied in relation to their effect on loading. It is found that very slow-moving traffic returns the highest loading events, rather than full stop conditions. The topic is especially relevant to existing bridges, where small differences in the loading may play an important role in the safety assessment and subsequent maintenance plans.

1 INTRODUCTION

1.1 *Motivation*

It is well acknowledged that long-span road bridges (about 50 m long and more) are governed by congested traffic rather than free-flow conditions. In free flowing traffic, vehicles have large gaps between them, while congestion implies long queues of closely spaced vehicles. Greater load effect results, even though there is no amplification for dynamic effects because of the slow speeds involved.

Although long-span bridges are strategic points of the road network, they are not well represented in current design codes due to their low number. For instance, Eurocode 1 (2003) applies only to spans up to 200 m. Common design practice usually relies on conservative assumptions about the traffic and does not consider variability of congestion patterns and driver behaviour. Since bridge maintenance operations are rather expensive, such assumptions may play a decisive role in the assessment of existing bridges. Therefore, a more accurate and site-specific traffic loading may result in significant savings in maintenance operations.

1.2 *Load models for long-span bridges*

The available models for traffic loading take into account the variability of truck weights, but they assume full stop traffic with a mix of cars and heavy vehicles at a bumper-to-bumper distance (Ivy et al. 1954, Buckland 1981, Flint & Neill Partnership 1986, Ditlevsen & Madsen 1994, Nowak & Luto-

mirska 2010). Truck weight data for these models comes either from traffic survey or, more recently, from weigh-in-motion stations. Other traffic data (such as average speeds or car counts) generally comes from embedded loop detectors, which may be combined with weigh-in-motion stations. Data is very often collected during free-flowing traffic, due to the fact that it occurs more frequently than congested traffic and the sensor accuracy is generally higher.

An important feature of traffic to bridge loading is that drivers do not usually like staying between larger vehicles and therefore cars (typically) move out from between trucks, as traffic becomes congested. This results in the formation of truck platoons in the slow lane, thereby changing the car-truck mix during congestion events. This makes the direct use of the widely available (and used) free traffic measurements problematic. As exceptions, Ricketts & Page (1997) and Nowak & Lutomirska (2010) used videos for collecting suitable congested data for bridge loading.

1.3 *Traffic micro-simulation*

Traffic micro-simulation (i.e. simulating the motion of individual vehicles) can adequately describe the interactions between vehicles, effectively generating different congestion patterns and car-truck mix. Notably, free traffic measurement can be used as initial traffic conditions for a micro-simulation.

More recently, the work of OBrien et al. (2010) has given a first approach to the use of micro-

simulation for bridge traffic loading. They have studied a heavy-congested Dutch long-span bridge and calibrated a commercial micro-simulation tool using WIM data, videos and strain gauge measurements.

Chen & Wu (2011) have used the cellular automaton approach (initially proposed by Nagel & Schreckenberg 1992), in which the bridge is divided into 7.5 m cell. However, the cellular structure does not allow for the variability of vehicle lengths and gaps between vehicles.

In this paper, the car-following Intelligent Driver Model is used (Treiber et al. 2000), coupled with the lane-changing model MOBIL (Kesting et al. 2007). The flow of two classes of vehicles (cars and trucks) running on a two-lane same-way road is studied. The different congested states are identified and the total load on a 100-m long bridge is calculated for each of these traffic histories. The simulations are carried out by means of an in-house program called *Simba* (Simulation for Bridge Assessment).

A similar study for single-lane and identical vehicles has been presented in Lipari et al. (2010).

2 MICRO-SIMULATION

2.1 Introduction

Micro-simulation models divide into car-following (single-lane) models and lane-changing (multi-lane) models. Micro-simulation has been widely used in traffic engineering and many models have been developed in the past decades, ranging in levels of complexity and accuracy (Brackstone & McDonald 1999, Orosz et al. 2010). Micro-simulation allows the study of the interaction between vehicles, as opposed to macro-simulation, which describes the traffic in terms of aggregate quantities such as flow and density.

Micro-simulation results should be compared to microscopic data, such as trajectory data. However, suitable microscopic data is difficult to collect and therefore microscopic models have been often calibrated and validated at the aggregated level (Hidas & Wagner 2004), since such data is widely available. However, it is still almost prohibitive to collect data for analyzing lane-changing manoeuvres, where vehicle tracking becomes a necessary tool to capture the interaction between the many vehicles involved in every manoeuvre.

On these grounds, while car-following models are relatively more established, lane-changing behaviour is still under debate in the traffic engineering community. In fact, along with macroscopic models, some car-following models are used to describe the global effects of multi-lane traffic (Treiber et al. 2000, Schönhof & Helbing 2007). However, these models would not be suitable for capturing the inter-

actions between cars and trucks and the subsequent formation of truck platoons.

2.2 The IDM car-following model

The Intelligent Driver Model is a car-following model, which has a modest number of physically-meaningful parameters, is collision-free, and has proven good match with real macroscopic congested traffic (Treiber et al. 2000, Helbing et al. 2009). It has also been calibrated with real trajectory data (Kesting & Treiber 2008, Hoogendoorn & Hoogendoorn 2010, Chen et al. 2010) and compared to other calibrated car-following models, returning results comparable to more complex models (Brockfeld et al. 2004, Punzo & Simonelli 2005).

In order to carry out the traffic micro-simulation, the in-house program *Simba* is used here. *Simba* implements the Intelligent Driver Model, as well as the lane-changing MOBIL model.

The IDM simulates driver behaviour in time through 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; $v(t)$, the current speed; $s(t)$, the current gap to the vehicle in front, and; $s^*(t)$, the minimum desired gap, given by:

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

in which, s_0 is the minimum bumper-to-bumper distance; T , the safe time headway; $\Delta v(t)$, the velocity difference between the current vehicle and the vehicle in front, and; b , the comfortable deceleration. Note that, when the front vehicle is faster, the desired minimum gap s^* in (2) can turn negative, generating an inconsistent driver behaviour (Caprani et al. 2011). Therefore, the desired minimum gap s^* is down-capped to the minimum bumper-to-bumper distance s_0 .

There are five parameters in this model to capture driver behaviour, which are relatively easy to measure. For simulation purposes, the length of the vehicle must also be known.

2.3 The MOBIL lane-changing model

The MOBIL lane-changing model has been proposed in Kesting et al. (2007). MOBIL can be adapted to symmetric and asymmetric passing rule. With the symmetric passing rule, vehicles can overtake from any side, whereas with the asymmetric rules, vehicles must use only the designated lane for overtaking. In the following, we use the symmetric passing rule. It must be noted that, while the asym-

metric passing rule would match the European motorway regulations, the symmetric rule seems more suitable for simulating driver behaviour during congested traffic.

The topology of a lane change event is illustrated in Figure 1 where the subscript c refers to the lane-changing vehicle, o refers to the old follower (in the current lane) and n to the new one (in the target lane). The tilde identifies the situation after the lane change. The front vehicles play a passive role, representing a “constraint” which affects the acceleration of the lane-changing vehicle. All the accelerations, current and proposed, are calculated according to the car-following model given in Equations (1) and (2).

Figure 1. Vehicles involved in lane-changing manoeuvre (adapted from Kesting et al. 2007)

A lane change occurs if both the *incentive* and the *safety criteria* is fulfilled. The incentive criterion is expressed as follows:

$$\tilde{a}_c(t) - a_c(t) = \Delta a_{th} + p \left[(a_n(t) - \tilde{a}_n(t)) + (a_o(t) - \tilde{a}_o(t)) \right] \quad (3)$$

This means that the acceleration advantage to be gained by lane-changing, $\tilde{a}_c - a_c$, must be greater than the sum of:

- the acceleration threshold Δa_{th} , which prevents overtaking with a marginal advantage, and
- the imposed disadvantage to followers in the target lane $a_n - \tilde{a}_n$ and in the old lane $a_o - \tilde{a}_o$, weighted through a politeness factor p .

Driver aggressiveness can be adjusted through the politeness factor. While $a_n - \tilde{a}_n$ is usually positive, representing for the new follower, n , a real disadvantage, $a_o - \tilde{a}_o$ is usually negative, representing for the old vehicle, o , an advantage. Thus, it is taken into account that a faster follower can pressurize its leader.

The strategy of Equation (3) gives name to the MOBIL acronym (Minimizing Overall Braking Induced by Lane changes).

The *safety criterion* limits the imposed deceleration to the follower in the target lane:

$$\tilde{a}_n(t) \geq b_{safe} \quad (4)$$

However, the safety criterion of Equation (4) seldom applies, as long as the politeness factor p is not too close to zero.

Asymmetric passing rules are easily implemented through the introduction of a bias acceleration Δa_{bias} towards the slow lane.

3 CONGESTED TRAFFIC STATES

3.1 Inducing congestion

Treiber et al. (2000) have shown that congestion can be effectively generated by applying flow-conserving inhomogeneities. They consist in a local variation of the parameters, by either decreasing locally the desired speed, v_0 , or increasing the safe time headway, T . It has been also shown that such local parameter variations act as an equivalent on-ramp bottleneck. This procedure has been successfully applied to *single-lane* simulations for simulating congested traffic on some *multi-lane* German motorways. However, there are no applications of combined IDM and MOBIL simulations for studying different congestion patterns. Therefore the following results may be of interest to the traffic engineering community as well.

In this paper, inhomogeneity is generated by increasing the safe time headway, T , downstream to, say, T' , which Treiber et al. (2000) state to be more effective than decreasing v_0 .

3.2 Bottleneck strength

A bottleneck strength, ΔQ , can be defined as the difference between the outflow, Q_{out} , with the original parameter set and the outflow, Q'_{out} , with the modified safe time headway T' .

$$\Delta Q(T') = Q_{out}(T) - Q'_{out}(T') \quad (4)$$

According to Treiber et al. (2000), the outflow Q_{out} is the *dynamic capacity*, i.e. the outflow from a congested state. It is different from (and smaller than) the static capacity Q_{max} , which can be attained only in free equilibrium traffic. However, for the parameter set chosen, its value is very close to the capacity of the road Q_{max} .

3.3 Congested traffic states

Depending on the inflow Q_{in} and the bottleneck strength ΔQ , the downstream traffic can take up any of the identifiable traffic states listed in Table 1. A combination of these congested states may also occur and these depend on the previous traffic history as well.

Table 1 – 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

4 MODEL AND SIMULATION PARAMETERS

4.1 Traffic stream

For this study, the vehicle stream is made up of two vehicle classes: cars and trucks. The parameters for each class are shown in Table 2. The car-following parameters are based on those used in Treiber et al. (2000), while the lane-changing parameters are chosen to match the (few) lane-change observations available in literature for two-lane motorways (Sparmann 1979, Yousif & Hunt 1995). Trucks have a smaller desired speed and are longer. All the parameters are constant, except for the desired speed, which is uniformly distributed, as done in Kesting et al. (2007).

Table 2 - Model parameters of IDM and MOBIL

	Cars	Trucks
Desired velocity, v_0	120 km/h ($\pm 20\%$)	80 km/h ($\pm 20\%$)
Safe time headway, T	1.6 s	1.6 s
Maximum acceleration, a	0.73 m/s ²	0.73 m/s ²
Comfortable deceleration, b	1.67 m/s ²	1.67 m/s ²
Minimum jam distance, s_0	2 m	2 m
Vehicle length, l	4 m	12 m
Politeness factor, p	0.2	0.2
Changing threshold, Δa_{th}	0.4 m/s ²	0.4 m/s ²
Maximum safe deceleration, b_{safe}	4 m/s ²	4 m/s ²
Bias for the slow lane, Δa_{bias}	0 m/s ²	0 m/s ²
Gross Vehicle Weight	20 kN	432 kN

Trucks comprise 20% of the total flow and are randomly injected between cars in the slow lane. Following this, the symmetric passing rule seems able to reproduce European congested traffic states, since most trucks remain on the slow lane. A standard truck was assumed (Grave et al. 2000), shown in Figure 2, with total weight taken as a common European highway maximum (44 t). Front and back overhangs of 0.9 m are also considered.

Figure 2. Assumed standard truck configuration.

4.2 Road geometry and bottleneck strengths

A two-lane same-direction 5000 m long road is considered. For the parameters chosen, the capacity Q_{max} is 1750 veh/h/lane, while the dynamic capacity Q_{out} is 1685 veh/h/lane.

Two inflows Q_{in} have been considered: a high inflow equal to the dynamic capacity $Q_{in} = Q_{out} = 1685$ veh/h/lane and a lower inflow of 1200 veh/h/lane (about 70% of the capacity). No on- or off-ramps are included, so the lane changes can be classified as discretionary, i.e. a vehicle does not need to follow a specific path, but only the most advantageous condition.

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' . Six different values of T' (1.9, 2.2, 2.8, 4.0, 6.4, and 11.2 s) are considered for the simulations, each of which is 1 hour long. These values are chosen in order to generate a wide range of congestion types. All the vehicles have an initial velocity of 54 km/h.

For each pair of Q_{in} and T' , eight one-hour simulations have been run, in order to account for the randomness involved in the truck injection and vehicle desired speed. Figure 3 shows the relation between the applied inhomogeneity $\Delta T = T' - T$ and the resulting bottleneck strength ΔQ . These results agree with those of Treiber et al. (2000).

Figure 3. Relation between the applied inhomogeneity ΔT and the bottleneck strength ΔQ .

For further comparison with the available traffic loading models, the full stop condition is also considered (FS). It corresponds to $\Delta T = \infty$ or $Q'_{out} = 0$ veh/h/lane. Then $\Delta Q = Q_{out} = 1685$ veh/h/lane, according to (4). Again, eight simulations are run for each inflow. In total, 112 one-hour simulations are run and analysed.

5 TRAFFIC RESULTS

5.1 Introduction

In this section, we analyse the results in terms of congested traffic states and lane change rate. Since traffic is always uncongested within the inhomoge-

neity (although with reduced flow), the results are shown for the section 0-3500 m.

5.2 Spatio-temporal congestion patterns

In order to identify the different congestion states, the following graphs show the spatio-temporal plots of the space mean speed for the inflow $Q_{in} = 1685$ veh/h/lane. The space-mean-speed is calculated as the harmonic mean of the individual speed crossing a point. A virtual detector is placed every 500 m and aggregates the individual speeds over 1 min, similarly to actual traffic sensors. The speeds axis is plotted upside down, so that the peaks represent congestion.

Figure 4 shows a SGW state, which results from $\Delta Q = 122$ veh/h/lane. It can be seen that the waves propagate backwards and the traffic is essentially free between the peaks, with speeds often exceeding 60 km/h. Observations from real traffic point out that the wave propagation speed is remarkably constant, with a typical value of -15 km/h, while there is no typical wavelength or period (Schönhof and Helbing 2007). Such a feature can be used for validating models describing the spatio-temporal congested traffic states (Treiber & Kesting, in press). In our simulations, the propagation speed is about -12 km/h, since the wavelength is about 1 km and its period is about 5 min. The small discrepancy in the propagation speed is probably due to the multi-lane parameter set chosen, since the car-following parameter set used here is taken from Treiber et al. (2000), who successfully reproduced the typical wave propagation speed of -15 km/h by means of single-lane simulations.

Figure 4. Spatio-temporal speed of stop-and-go waves ($Q_{in} = 1685$ veh/h/lane; $\Delta Q = 122$ veh/h/lane).

Figure 5 shows an OCT state, which results from $\Delta Q = 334$ veh/h/lane. It is characterised by a rough surface, rather than spiky peaks. The speed between the peaks rarely exceeds 40 km/h.

Figure 6 shows a combined state of HCT near the inhomogeneity and OCT behind, which results from $\Delta Q = 602$ veh/h/lane. It is equivalent to an on-ramp flow of 1204 veh/h, similar to a busy one-lane slip

road onto a motorway. The space average speed around the HCT area is 14.3 km/h.

Finally, Figure 7 shows a full HCT state, which results from $\Delta Q = 1161$ veh/h/lane. This is a heavy form of congestion, higher than the bottleneck strength range considered in Treiber et al. (2000). It may be related to a lane closure due to an accident. The average speed on the congested area is 5 km/h.

Figure 5. Spatio-temporal speed plot of oscillating congested traffic ($Q_{in} = 1685$ veh/h/lane; $\Delta Q = 334$ veh/h/lane).

Figure 6. Spatio-temporal speed plot of combined homogeneous and oscillating congested traffic ($Q_{in} = 1685$ veh/h/lane; $\Delta Q = 602$ veh/h/lane).

Figure 7. Spatio-temporal speed plot of homogeneous congested traffic ($Q_{in} = 1685$ veh/h/lane; $\Delta Q = 1161$ veh/h/lane).

In the case of $Q_{in} = 1200$ veh/h/lane, since the traffic is not at capacity, we need to increase the bottleneck strength up to 602 veh/h/lane, in order to obtain congestion. Here, we do not find the oscillating congestion state, but we move straight to an HCT/OCT state. This result agrees with the findings of Treiber et al. (2000). Afterwards, we find an HCT state. The HCT/OCT and HCT states grow less rapidly but are qualitatively similar to the ones depicted in Figures 6 and 7.

5.3 Lane change rate

Figure 8 shows the variation of the lane change rate against the bottleneck strength. Since the number of lane changes during the congestion is very low, the lane change rate is computed over the first 30 minutes of each simulation. Doing so captures vehicle lane change activity as vehicles approach the congestion front.

Figure 8. Average lane change rate for different bottleneck strengths and inflows.

As expected, the lane change rate drops with increasing bottleneck strengths, since the available gaps reduce in size. The trend is sharper for the inflow $Q_{in} = 1200$ veh/h/lane, while the high inflow $Q_{in} = 1685$ veh/h/lane allows little space for overtaking even in free traffic.

The lane change rates during free traffic have been used to calibrate the lane-changing parameters (Table 2), in order to make it similar to available field observations for two-lane motorways (Sparmann 1979, Yousif & Hunt 1995).

6 LOAD EFFECT RESULTS

6.1 Introduction

In this section the load effect induced by the different congested traffic states on a sample bridge is studied to identify the most critical congestion states for bridge loading. A 100 m long bridge is centred at 3000 m. For each one-hour simulation, the maximum total load on the bridge is computed.

Assuming no cars and bumper-to-bumper distance, the maximum possible total load on the bridge is 6160 kN (7 trucks in each lane), given the constant gross vehicle weights of Table 2.

6.2 Load effects for high inflow

Figure 9 shows the total load on the bridge against the bottleneck strength for the inflow $Q_{in} = 1685$ veh/h/lane.

Figure 9. Maximum hourly total load for different bottleneck strengths on the sample bridge ($Q_{in} = 1685$ veh/h/lane).

It can be noted that already a slight congestion (SGW, $\Delta Q = 122$ veh/h/lane) causes a load almost double than in free traffic. Then, the load effect increases in a quasi-linear fashion until the HCT point corresponding to $\Delta Q = 1359$ veh/h/lane, which represents a very heavy congestion. However, given that the position of bridge is close to the downstream congestion front, only the HCT state will affect the loading. Then, the maximum hourly load decreases when reaching the full stop point ($\Delta Q = 1685$ veh/h/lane). Notably, the average maximum load corresponding to the full stop is even lower than the average maximum for any other congestion type.

This finding is interesting, since previous load models used the full stop condition. Such a finding can be explained by considering that the heavier the congestion, the slower the speed and the closer the vehicles. On one hand, heavy congestion results in greater load effects (since vehicles are closely spaced), but on the other, fewer vehicles have the chance to cross the bridge. Therefore, the probability of having a critical loading event decreases. Figure 10 illustrates this finding, by comparing a load effect history for the full stop condition with the HCT state ($\Delta Q = 1161$ veh/h/lane).

Figure 10. Total load time-history of a full stop simulation and HCT state ($Q_{in} = 1685$ veh/h/lane).

6.3 Load effects for medium inflow

For the lower inflow, $Q_{in} = 1200$ veh/h/lane, the results are shown in Figure 11. As discussed previously, a larger bottleneck strength is required in order to induce congestion, which, on the bridge area, is made up of only HCT traffic. Again, the full stop traffic state load effect is, on average, lower than the slow-moving traffic. However, it shows a higher variation from the average value.

Figure 11. Maximum hourly total load for different bottleneck strengths on the sample bridge ($Q_{in} = 1200$ veh/h/lane).

6.4 Summary of findings

From the results presented it is clear that the heavy-congested and slow-moving HCT state is the most critical one for the bridge and traffic considered. Interestingly, this is in contrast to the full-stop state usually considered in developing loading models.

Further, the statistics depicted in Figure 12 show that there is little difference in the resulting load effects for the two different inflows, provided that the inflow is high enough to trigger congestion. Figure 12 also shows that it is the bottleneck strength that governs, rather than the inflow. This is an important finding, since periods of inflow lower than capacity (for instance, out of peak hours) can be potentially critical for bridges, by triggering HCT states.

Finally, we note that the results obtained are quite variable. The coefficients of variation of load effect range from 0.02 ($Q_{in} = 1685$ veh/h/lane, $\Delta Q = 602$

veh/h/lane, HCT/OCT) to 0.26 ($Q_{in} = 1200$ veh/h/lane, $\Delta Q = 1685$ veh/h/lane, FS). As a result, it is clear that many more simulations of further configurations are needed to draw more general conclusions about the nature of the resulting load effect.

Figure 12. Mean and standard deviation of the maximum hourly total load for each inflow and bottleneck strengths.

7 CONCLUSIONS

This paper investigates the effect of different congestion patterns on the total load of a sample two-lane same-way long-span bridge, by using a multi-lane micro-simulation tool. Previous load models neglect different congestion patterns, assuming a stationary queue of vehicles.

The car-following model used here is able to reproduce observed congestion patterns. It is extended with a lane-changing model, necessary for simulating realistic car-truck mixes relevant to long-span highway bridge traffic loading.

We show that the bumper-to-bumper queue is not the most critical situation for the sample long-span bridge, since it does not allow the flowing of vehicles and therefore decreases the probability of observing critical loading events. Indeed slow-moving traffic, corresponding to heavy congestion, is the most critical state for the bridge. In this case, speeds are low enough (order of 5 km/h) to have the vehicles closely spaced, but it still allows the crossing of vehicles and formation of peak load effects.

It is also found here that it is the bottleneck strength that governs the traffic, rather than the inflow. At an inflow of about 70% of the road capacity, there are fewer traffic oscillations and the traffic tends to be homogeneously slow-moving. This implies that congestion arising outside of peak hours can be critical for bridge loading.

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