PROGRESSION OF DIFFERENT VEHICLE TYPES IN A SIGNALISED URBAN ARTERIAL CORRIDOR — MODEL DEVELOPMENT AND CALIBRATION

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Abstract: Increasing freight demand has lead to a greater presence of multi-combination-vehicles, such as B-doubles, on Australia's roads. The impacts of these vehicles on signalised urban arterial corridor performance are of concern to many road users. This paper describes the development and calibration of a microsimulation model to be used to study these impacts. The discrete-time model considers all vehicles to behave autonomously according to their own capabilities and in response to surrounding vehicles and traffic controls. Traffic flow and trajectory data from a major freight route through Brisbane's suburbs was used to calibrate the model. Different headway distributions were found for different vehicle types, and modelled by a lognormal distribution. A GPS-equipped car followed subject vehicles along the corridor, whilst the status of 20 signalised intersections was recorded by the STREAMS traffic management system. Acceleration profiles were used to calibrate the unimpeded acceleration models for different vehicle types.

Keywords: Signalised intersection, coordination, microsimulation, progression, heavy vehicle, B-double.

1. INTRODUCTION

Increasing demand for road freight has led to a widespread adoption of more-productive multi-combination vehicles (MCVs) by Australia’s road freight industry. The B-double is one type of MCV which has been granted access to urban road networks, further increasing its attractiveness for intra- and inter-urban operations. This increasing presence of MCVs would impact on urban arterial traffic corridor performance – individual vehicles are heavier and longer, but fewer vehicles are required to perform a given freight task.

1.1. Objective

This study aims to quantify the traffic-related impacts of MCVs in urban arterial corridors, and to assess various traffic management scenarios for productivity gains. To achieve this, a microsimulation-level model of traffic flow in an urban arterial corridor has being developed. This paper describes the development and calibration of that model.

1.2. Background

A B-double is a relatively common type of heavy vehicle in Australia comprising a prime mover (tractor) towing two semi-trailers, the front of the second trailer being supported by the rear of the first semi-trailer. Typically up to 25 metres in length and weighing up to 62.5 tonnes on nine axles, B-doubles are longer and heavier than general access vehicles which are up to 19 metres in length and weigh up to 42.5 tonnes on six axles. The Survey of Motor Vehicle Usage (ABS 1972-2004) has recorded a steady increase in the use of B-doubles, from 14 per cent of total freight vehicle-kilometres travelled (VKT) in 1998 to 24 per cent in 2003. This accounted for almost all the increase in freight VKT over this period.

In recognition of this changing traffic composition, the appropriateness of factors used to account for heavy vehicles in existing models may be questioned. Both the Highway Capacity Manual (HCM, Transportation Research Board 2000) and ARRB’s ARR123 (Akçelik 1981) require that saturation flow rates at signalised intersections be reduced by a factor that is dependent on the proportion of heavy vehicles and a Passenger Car Equivalence (PCE) value representing an aggregation of heavy vehicle types. These sources recommend that a PCE value of 2 passenger cars per heavy vehicle for through movements. This value can be traced back to the 1960’s (Miller 1968), which used vastly different vehicles to those found on modern roads.

Several researchers have identified this inadequate characterisation of heavy vehicles at signalised intersections. These include finding that PCEs of larger (but not smaller) heavy vehicles depend upon queue position (Molina 1987), and that heavy vehicles were major contributors to increased headways particularly with turning movements (Cuddon and Ogden 1992). Haldane and Bunker (2002) found PCE values for a range of MCVs; concluding that the traffic efficiency, in terms of payload per PCE, was greater for MCVs than for other freight vehicles. Ramsay *et al*. (2004) showed that a trajectory diagram can be used to model the effect of heavy vehicles on signalised intersection control delay, confirming Molina’s empirical findings of heavy vehicle queue position effects on saturation flow, and noting that following vehicles also incur a greater delay.

Signalised intersections generally do not operate in isolation, with traffic moving in platoons which are subject to dispersion. Heavy vehicles affect this dispersion through their generally inferior acceleration rate, splitting a platoon if positioned in the middle, or concentrating if positioned at the front. Overtaking opportunities serve to decrease the effect of heavy vehicles on urban traffic flow. A greater understanding of these mechanisms can be obtained through the development and application of a microsimulation corridor traffic progression model.

2. MODEL DEVELOPMENT

The model developed is considered to be a discrete-time heterogeneous-traffic corridor-level microsimulation model. Similar to other microsimulation models, it considers a number of vehicles at various positions and speeds along the corridor. Each vehicle behaves autonomously in response to its own capabilities, to surrounding traffic, and to traffic control measures such as traffic signals and speed limits.

2.1. Kinematics

Being specifically intended for studying the effects of heavy vehicles, considerable emphasis is placed on the kinematic characterisation of each vehicle. Several authors have proposed various models of for this purpose, ranging from assuming constant acceleration (for example MRWA 1992) through to detailed calculation of the tractive and resistive forces acting on the vehicle (Rakha and Lucic 2002). Akçelik and Biggs (1987) empirically modelled acceleration as a polynomial function of elapsed time, accurately showing the increase and then decrease of acceleration as a vehicle starts from rest. Shekleton (2002) modelled acceleration as a polynomial function of speed, setting the initial acceleration to zero. Neither of these last two models can be readily implemented in a discrete time-step numerical integration routine since, in setting the initial acceleration to zero, the simulation can not move off from a state of rest.

Although appropriate modelling of fuel consumption and emissions, the level of detail used in these above models is considered excessive for the current application, where the primary outputs are vehicle position and speed at a given time.

Bunker and Haldane (2003) developed a simple relationship for heavy vehicle acceleration which decreased linearly with elapsed time, based on testing under controlled conditions of a number of multi-combination vehicles. An empirical relationship based on elapsed time is well suited to unimpeded cases, for example in calculating intersection or railway level crossing times, however it is less suited to the general case where a vehicle's motion is also influenced by surrounding vehicles and traffic control.

The current model assumes that a vehicle's unimpeded acceleration rate decreases linearly with increasing speed. This formulation has been widely used (for example Long 2000) and, not depending on elapsed time, offers advantages in microsimulation models. Relatively simple expressions for speed and distance can be obtained by integration.

2.2. Car (and truck) following

Vehicle motions are also dependent upon those of surrounding vehicles and traffic control. A car following model described in Cohen (2002) has been implemented in the model, although alternative models may be substituted. Vehicles keep track of the vehicle in front (which may change during the simulation). Vehicles accelerate or decelerate so as to maintain a specified headway to the lead vehicle, and to be able stop safely if the lead vehicle does so. This acceleration is limited to not exceed the vehicle's acceleration capability (see above), the current speed limit, or the braking capability.

2.3. Lane changing

The ability for vehicles to change lanes during a simulation is an essential component of a multi-lane corridor model. This is particularly so for a study of the traffic-related effects of heavy-vehicles. The model currently does not permit vehicles to change lanes so, in its present form, would be expected to overestimate their impacts. Various models (for example Gipps 1986; Hidas 2002) have been proposed for lane-changing in microsimulation traffic models, all of which keep track of the surrounding vehicles as the simulation proceeds. Discretionary lane-changing, as occurs when avoiding a slower vehicle ahead, is a three step process – identifying that a vehicle is travelling below its desired speed, waiting for an adequate gap, and handling the actual move.

2.4. Traffic controls

Vehicles respond to traffic signals by stopping if the signal is red and the vehicle is within a safe stopping distance which depends upon its speed. Traffic signals along the corridor operate at fixed offsets from adjacent signals when maintained within a given timing plan.

Speed limits may be dependent upon the distance along the corridor, with vehicles not being able to accelerate past the speed limit and coasting down to reach a lower speed limit. The gradient of the road may also be varied with distance, with grade affecting the maximum acceleration rate and maximum speed of different vehicle types.

2.5. Assigning characteristics

Vehicles are assigned specific vehicle types when they are initialised. Each vehicle type has different characteristics – trucks being longer and having lower acceleration than cars. Headways between vehicles are also able to be varied according to specified distributions.

The model is currently implemented as a macro-based spreadsheet application, with different pages for road, traffic and vehicle data input. The model produces distance-time plots for all vehicles, from which specific output measures may be calculated.

The model contains three areas which require calibration: The types of vehicles and headways between vehicles must be specified to ensure that an appropriate traffic mixture is used as input to the simulation. Vehicle-type specific kinematic parameters must be specified to determine how vehicles move through the corridor – for a study of heavy vehicle effects this is very important. Corridor geometry and speed environments must be also specified to account for these limits to vehicle motions.

3. TEST PROGRAM

A major freight route through suburban Brisbane was chosen as the test site. Forming part of a major freight route between an industrial area and intermodal terminal and the Port of Brisbane, the 11-kilometre long corridor runs east-west between two motorways through varying terrain (Figure 1). It passes through residential and commercial areas, offering a good opportunity to study the interaction between vehicle types under different traffic conditions. There are 20 intersections along the corridor, all being controlled and monitored from the STREAMS traffic management system.

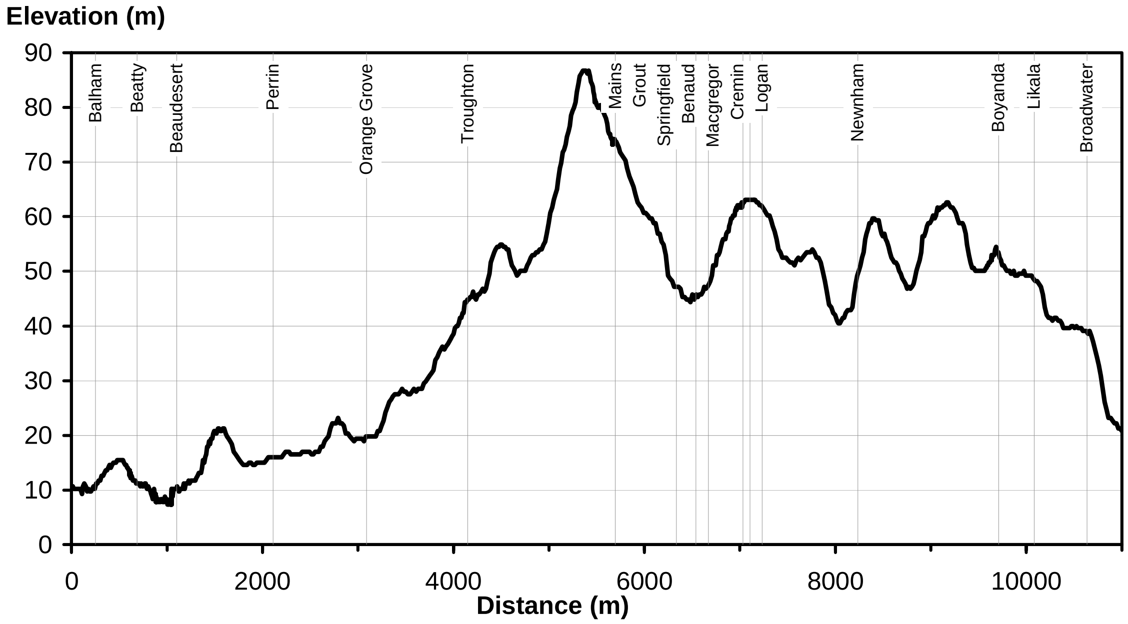

Figure 1 – Vertical elevation profile of the test corridor

Two data collection exercises were undertaken in September 2004 to calibrate the traffic model. A traffic counter was installed at one location to determine the proportion of each vehicle type and the headways between vehicles. A floating car survey was undertaken with a GPS-equipped surveillant chase car in order to observe vehicle-specific kinematic parameters and to collect road geometry data.

3.1. Traffic counts

Traffic was assumed to consist of the following five vehicle types: passenger cars, 4-wheel drives and light commercial vehicles, rigid trucks, articulated trucks, and B-doubles. It should be noted that the traffic counter was unable to differentiate between passenger cars, 4-wheel drives and light commercial vehicles, as they all have similar wheelbases.

Two automatic traffic counters were used to record individual axle detections at a mid-block location on the test corridor. The traffic counter software was able to report individual vehicle records; however it did not offer adequate time resolution for accurate calculation of headway distributions and was found to misclassify some vehicles. A program was written to convert the axle detection into individual vehicle records, reporting times to the required accuracy and minimising misclassifications. Figure 2 shows an extract from the processed vehicle file, showing ten short vehicles, two articulated vehicles and one B-double.

```
___DATE___ ____TIME___ LA CL AXGP KM/H  H/WAY  OAWB
2004/09/13 19:53:33.093 2  1 0202 77.20 0.999  2.79
2004/09/13 19:53:40.226 2  1 0202 93.44 7.133  2.36
2004/09/13 19:53:43.326 1  9 0603 70.91 11.12 13.23 o oo    ooo
2004/09/13 19:53:45.122 2  1 0202 83.43 4.897  2.26
2004/09/13 19:53:58.080 2  1 0202 64.87 12.96  2.39
2004/09/13 19:53:58.082 1 10 0904 68.50 14.76 21.51 o oo    ooo    ooo
2004/09/13 19:54:03.203 2  1 0202 81.16 5.123  2.35
2004/09/13 19:54:12.292 1  1 0202 71.91 14.21  2.59
2004/09/13 19:54:14.343 2  1 0202 77.79 11.14  2.47
2004/09/13 19:54:16.848 1  9 0603 70.96 4.556 14.22 o  oo     ooo
2004/09/13 19:54:55.354 2  1 0202 84.08 41.01  2.44
2004/09/13 19:54:57.150 1  1 0202 75.04 40.30  2.46
2004/09/13 19:54:58.879 1  1 0202 70.18 1.729  2.45
```

Figure 2 – Extract from processed traffic counter file

A total of 65,110 vehicles were extracted from the axle detection records, 99.9% of which were able to be classified into one of the 12 Austroads vehicle classification bins. These 12 vehicle classifications were then aggregated into four vehicle types, corresponding to short vehicles, rigid trucks, articulated trucks and B-doubles. Headways between consecutive vehicles were calculated, enabling distributions to be ascertained for each vehicle type.

3.2. Floating car survey

Ideally, the travel characteristics of a vehicle are best measured by directly measuring the position of that vehicle. However, drivers of the test vehicles would generally be aware of the presence of the experiment, which in turn may influence their behaviour. In order to avoid this, a surveillance car fitted with GPS instrumentation followed behind the subject vehicle as it was driven over the corridor. This method offers two main advantages in that the driver of the subject vehicle generally would not be aware of or be influenced by the experiment, and that a greater number of test runs can be conducted without having to identify consenting subjects and having to fit instrumentation to different vehicles. The obvious disadvantage is that the leading car’s motion cannot be exactly replicated, and may be lost in traffic (particularly around traffic signals).

Modern handheld GPS receivers considerably simplify the task of recording vehicle trajectories. The GPS receiver was set to record position, speed and altitude at one-second intervals. An external antenna was mounted on the vehicle’s roof to improve satellite reception. GPS is frequently quoted (for example Garmin 2003) as being able to resolve absolute positions to a 15-metre radius without resorting to differential GPS systems. In use, the GPS was found to have a much better resolution than this, particularly when calculating relative distances. Speed resolution is quoted as 0.1 knot (0.185 km/h), however it is only reported to two significant digits. A laptop computer was used to retrieve data from the GPS receiver when its internal memory reached capacity. This occurred after 10,000 seconds, giving time for 8 runs of the corridor between downloads.

A total of 126 runs of the test corridor were recorded, following a range of vehicle types in both eastbound and westbound directions. The type of vehicle being followed, travel time, average speed and number of stops were recorded for each run.

At the same time, the status of each of the 20 intersections along the corridor was recorded by the STREAMS traffic management system. A few days after each survey, the intersection status files were downloaded from STREAMS. This enabled the calculation of movement start and finish times as the subject vehicle was driven past each intersection.

This can be shown graphically in Figure 3, where the vehicle's trajectory is superimposed on the corridor timing chart. The vehicle can be seen to stop for red traffic signals (indicated by horizontal lines), and accelerates when the signal changes to green. The articulated truck being followed in this example turned off the corridor towards the end of the run, and the instrumented car can be seen to take a shorter time to reach the free speed than when it was following the truck. Figure 4 shows a graph of speed and number of stops against distance for the same run.

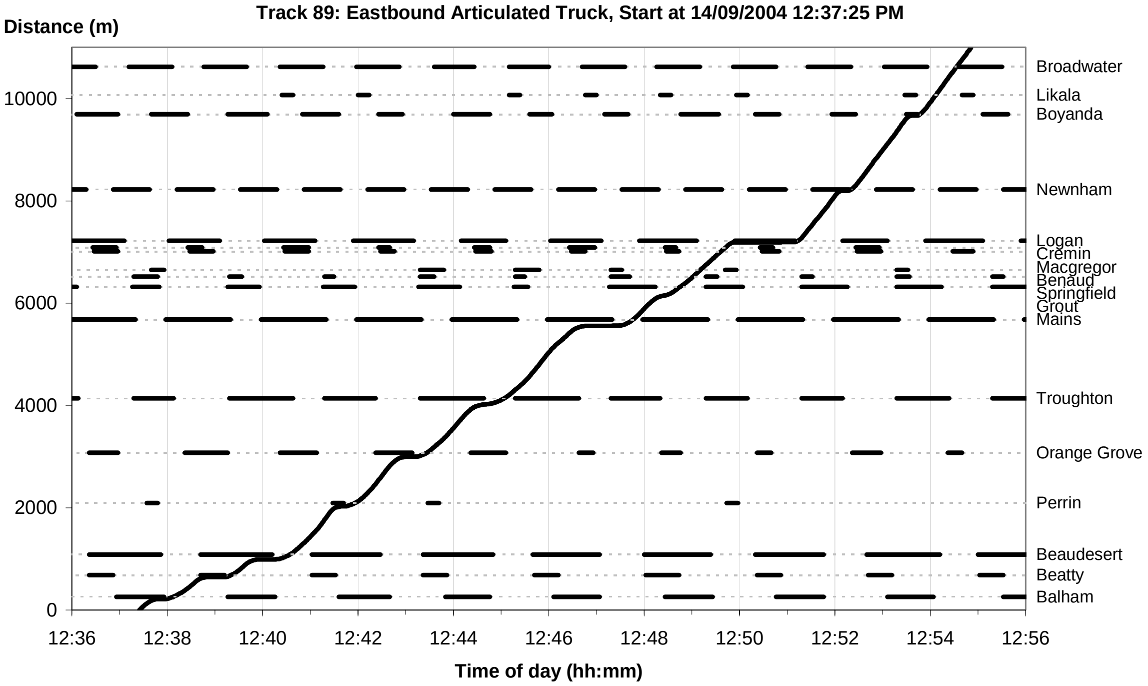

Figure 3 – Vehicle trajectory (GPS data) superimposed on the corridor timing diagram

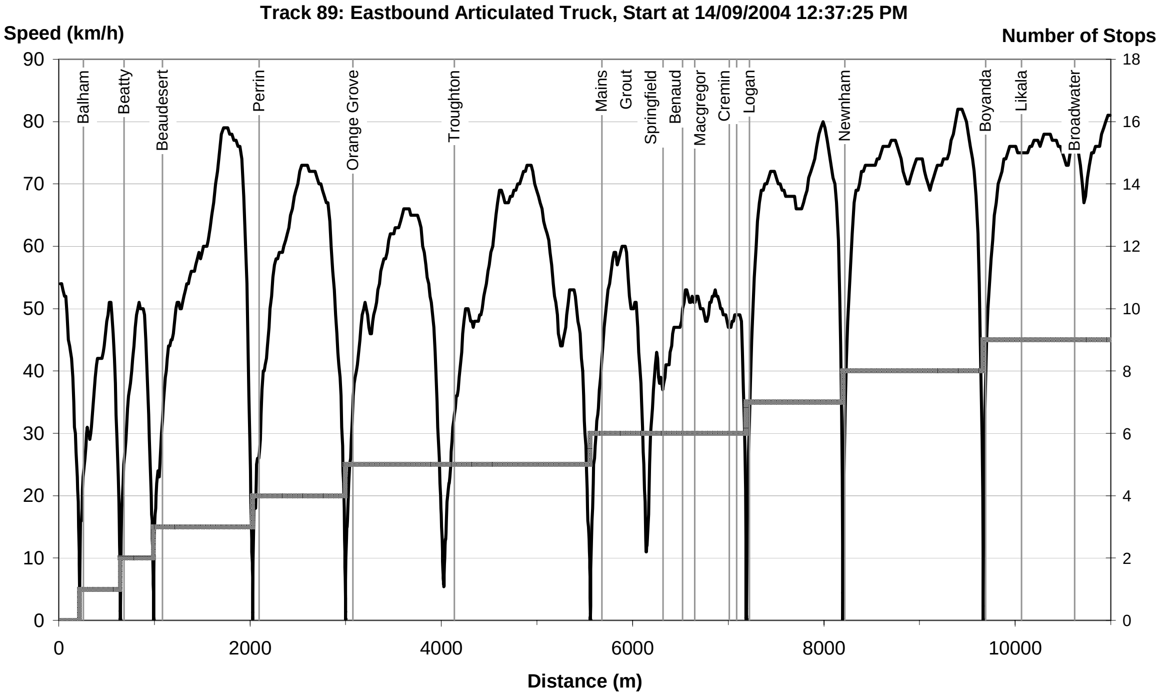

Figure 4 – Typical speed and stopping profile from GPS data

4. ANALYSIS

4.1. Vehicle type characterisation

The number and proportion of each vehicle type are shown in the last two columns of Table 1. About 13.6 per cent of vehicles were classified as a truck of one type or another. Considering the undetermined number of light commercial vehicles; the total proportion of commercial vehicles on the corridor would be expected to exceed 15 per cent.

Table 1 – Distribution of vehicle types

Following Vehicle Type	*Leading Vehicle Type*									
	Short Vehicle		*Rigid Truck*		*Articulated Truck*		*B-double*		*All Vehicles*	
	Number	*%*	*Number*	*%*	*Number*	*%*	*Number*	*%*	*Number*	*%*
Short Vehicle	49 345	87.7	4 013	79.3	2 233	75.9	606	76.9	56 249	86.4
Rigid Truck	3 996	7.1	602	11.9	361	12.3	87	11.0	5 058	7.8
Articulated Truck	2 246	4.0	351	6.9	270	9.2	69	8.8	2 941	4.5
B-double	609	1.1	76	1.5	75	2.6	26	3.3	788	1.2
Total	**56 249**	**100.0**	**5 058**	**100.0**	**2 941**	**100.0**	**788**	**100.0**	**65 110**	**100.0**

Table 1 also shows a dependence of vehicle type upon the preceding vehicle type. By comparison of the columns, trucks (particularly articulated trucks and B-doubles) are more likely than short vehicles to be followed by other trucks. This may be due to the lower acceleration capability of trucks compared to cars – when a truck arrives behind an even slower truck; it is less able than a car is to overtake the slow leading vehicle.

4.2. Headway characterisation

Headways between consecutive vehicles were extracted from the traffic count data. The cumulative distribution plot (Figure 5) shows a relationship between vehicle size and headway, with larger vehicles generally travelling at a greater headway to the preceding vehicle than smaller vehicles do.

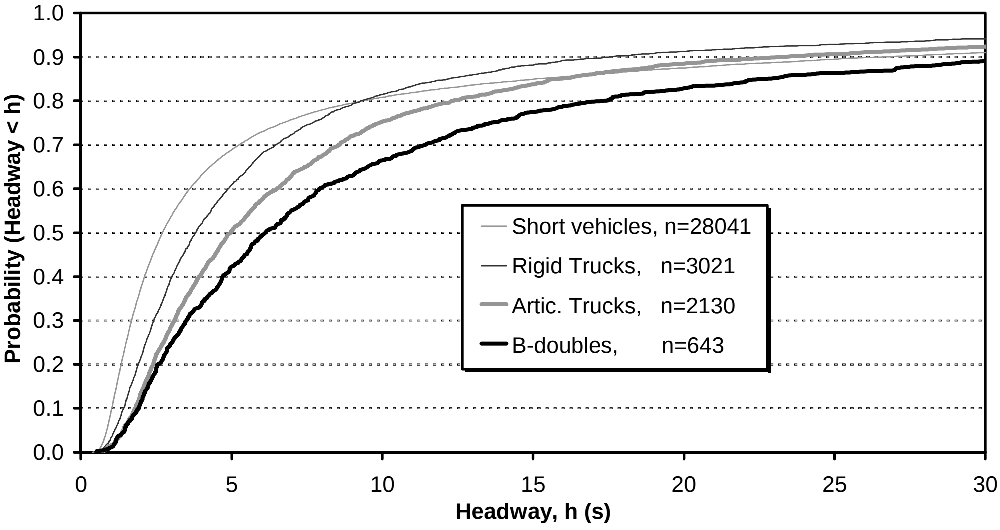

Figure 5 – Cumulative probability distribution of headways, Lane 1

Several probability distribution functions were fitted to the headway distribution data to ascertain the best distribution to use. A Cowan's (1975) M3 distribution has frequently been used (for example Sullivan and Troutbeck 1994) to characterise vehicle headways where a proportion of vehicles are travelling at a minimum headway and the others follow an exponential distribution. However, a lognormal distribution was found to provide a better fit to the data for all vehicle types in both lanes.

Figure 6 shows the lognormal probability density functions which were fitted to the headway data for each vehicle type in lane 1 (left lane). Larger vehicles tend to have greater minimum headways and mean headways. Minimum headways are much lower when considering both lanes, as vehicles can travel beside each other.

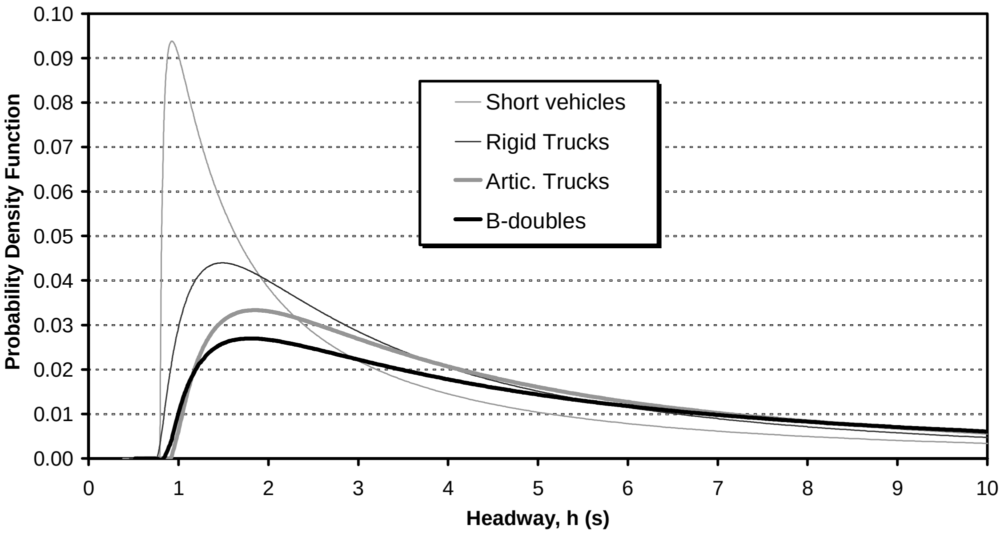

Figure 6 – Lognormal headway probability density functions, Lane 1

4.3. Acceleration characterisation

The acceleration of vehicles is the key to the simulation model – both the speed and distance travelled depend on appropriate representation of a vehicle's acceleration as it moves along the corridor. Acceleration is modelled as decreasing linearly with speed, as given by Eqn (1). This equation may be integrated to give expressions for speed and distance as functions of time.

$$a(t) = \begin{cases} \alpha\left(1 - g(x)/g_{\max}\right) - \beta v(t) & v(t) < v_{\lim} \\ 0 & v(t) = v_{\lim} \\ d_c & v(t) > v_{\lim} \end{cases} \tag{1}$$

where $a(t)$ is the instantaneous acceleration at time t;
$v(t)$ is the instantaneous speed;
$g(x)$ is the grade under the vehicle, a function of distance x;
$v_{\lim}$ is the current speed limit, a function of distance;
α is the maximum acceleration on level ground, occurring when $v = 0$;
β is the rate of decrease of acceleration with increasing speed;
$g_{\max}$ is the maximum grade which the vehicle can ascend; and
d_c is the deceleration used when coasting down from above the speed limit.

To calibrate the model given in Eqn (1), the speed / distance chart (Figure 4) was examined for each run along the corridor. Times when subject vehicles accelerated (without obstruction) from rest near an intersection stop bar to the free speed were identified, and then model parameters α and β determined to match both the distance and speed reached after the elapsed time. There may appear to be a large amount of scatter in the acceleration data, as shown in Figure 7 (left), due to the following vehicle being unable to exactly replicate the trajectory of the lead vehicle and the truncation of speed displayed by the GPS receiver. However, when the acceleration is integrated twice to give distance (Figure 7 right), these discrepancies are much less obvious.

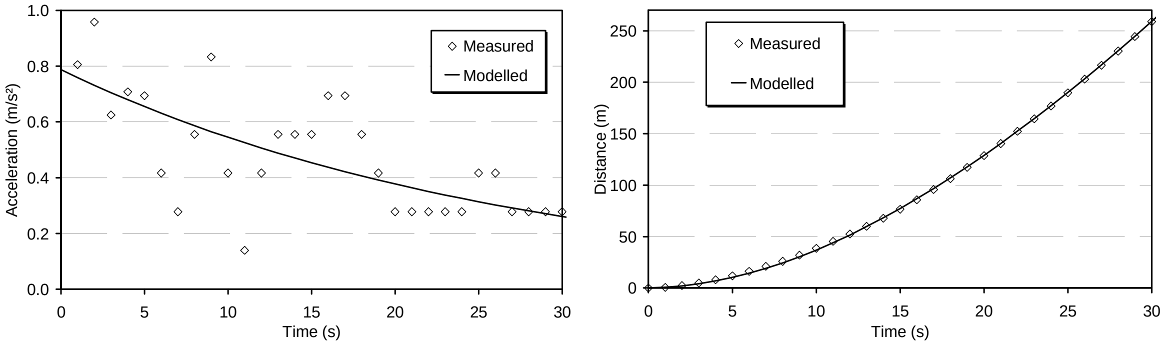

Figure 7 – Acceleration (left) and distance (right) during intersection queue departure.

These parameters were used to determine the time and distance taken to accelerate to a nominal speed (taken to be 60 km/h, the speed limit along much of the test corridor). Times and distances were averaged for each vehicle type, excluding those for which this nominal speed was close to the terminal speed, given by α / β. These averages were then used to determine generic parameters for each vehicle type.

A total of 344 acceleration traces were examined this way. A large variance in acceleration times and distances was found for the B-double vehicle type. This was able to be traced to runs at three particular intersections, Mains Road, Logan Road and Newnham Road, all of which had significant grades (Figure 1). The analysis was repeated without using data from these intersections, with Table 2 showing the acceleration times, distances and model parameters. The results show little difference in acceleration time or distance between articulated trucks and B-doubles; differences in performance on grade can be re-introduced by using different values of g_{max} in Eqn (1).

Table 2 – Acceleration profiles for each vehicle type

Vehicle Type	*Accelerating from rest to 60 km/h*				*Model Parameter*	
	Time (s)		*Distance (m)*			
	Mean	*Standard Deviation*	*Mean*	*Standard Deviation*	α *(m/s^2)*	β *(1/s)*
Passenger Car *(n=92)*	16.1	5.7	160	57	1.771	0.074
4WD / Light Commercial Vehicle *(n=34)*	20.2	6.9	205	68	1.501	0.067
Rigid Truck *(n=30)*	29.3	9.8	283	92	0.894	0.034
Articulated Truck *(n=28)*	34.6	8.9	334	97	0.753	0.028
B-double *(n=14)*	35.5	15.7	335	138	0.683	0.023

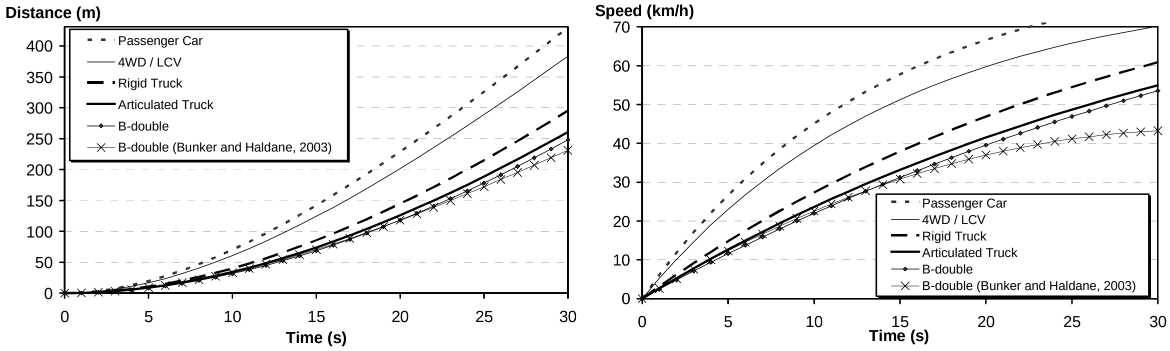

Figure 8 – Distance (left) and speed (right) of different vehicle types accelerating from rest

Figure 8 shows the distances and speeds predicted by the models plotted against time, confirming the performance differences between vehicle types. Included for comparison are Bunker and Haldane's (2003) results from controlled testing of a 62,100 kg B-double (maximum legal limit of 62,500 kg). Many of the vehicles in the current study were unladen or partially laden so, on average, vehicles would be expected to be capable of higher acceleration than that found by Bunker and Haldane.

The deceleration profiles recorded as vehicles brake when approaching a red or yellow traffic signal may be analysed using the same technique. Deceleration is less dependent upon vehicle type, with braking capabilities of larger vehicles increasing with increasing mass. Long (2000) recommends a deceleration rate for average motorists which is speed dependent (Eqn (1), with α = 3.0 m/s² and β = 0.133 s^{-1}). Calculations for stopping sight distance and for amber signal intervals commonly use a constant deceleration value of 3.0 m/s². In-service decelerations of 1.7 m/s² for cars, 1.5 m/s² for light trucks and 1.2 m/s² for heavy trucks were observed at intersections in New Zealand (Dibley and Reid 1990).

Intersection control delay is essentially independent of deceleration rate; heavy braking over a short distance having the same effect as light braking over a longer distance. What is affected is the decision whether to stop when approaching a yellow traffic signal.

5. PUTTING IT TOGETHER

A stochastic microsimulation traffic model requires several parameters to be calibrated to ensure that correct distributions are used for vehicle types, headways and lane allocations. Using the methods described in this paper, these parameters can be estimated from individual vehicle data collected by an automatic traffic counter. The motion of vehicle units during the simulation also require calibration, and an appropriate function for maximum vehicle acceleration can be determined from GPS data collected during a floating car survey. This survey also provides speed and stopping data for comparison to that predicted by the model. Knowledge of the status of intersections along the test corridor would also be required; this information could be obtained from a centralised traffic management system such as STREAMS.

Following further verification and validation, the model will be able to be applied to examine the effects of various traffic policy and priority issues on traffic flow through an urban arterial traffic corridor. It offers advantages in being able to accurately model vehicles with different characteristics in urban traffic corridor, without many of the overheads associated with full-scale commercial microsimulation software.

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