

Treating Inadequate Headway's on a High Flow Freeway

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Abstract: The effects of altering headway's on a high flow freeway are multifarious. Inadequate headway's cause flow instability and accidents. Increasing headway's can reduce flow and induce jamming, making determining the optimum headway difficult. Using a micro-simulation this paper determines the costs of implementing various headway treatment regimes and compares these to the likely benefits resulting from accident reduction. The most beneficial headway regime is then put forward.

In order to implement this treatment regime a roadside Intelligent Transport System (ITS) following-distance advisory and enforcement system is suggested. The system takes varying flow and weather conditions into consideration before modifying the headway's of drivers.

Key Words: Tailgating; Headway; Following; Safety; Accident

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1 Introduction

Most drivers underestimate the distance required to stop and follow too-closely, causing traffic waves and accidents. Traffic waves are generated by changes in speed that are accentuated when following too-closely. Traffic waves can cause upstream jamming as downstream speed variations get magnified while they travel along queues of traffic, forcing upstream traffic to brake suddenly to avoid an accident. These waves can cause accidents that create more jamming. Each accident causes significant delays for waiting drivers, often resulting in large scale jamming.

Inadequate following-distance accounts for between 30 and 40% of all freeway accidents, killing and injuring thousands of people every year throughout Europe, costing the European Community billions [14, 50].

It has been suggested by several sources using micro-simulations and empirical observations that increasing time headway will reduce the effect of density waves and accidents [8, 11, 47, 50, 55, 78]. Treiber and Helbing [63] found increasing the headway generally improves flow stability. Increasing headway in anticipative situations has been found to produce a forty-fold reduction in accidents in a simulation designed to model freeway flows with accidents [79]. The highest flow-state has been found to be the synchronous/critical-density flow-phase that is on the limit of flow breakdown. This phase is also where accidents are the most common. Driver's slow reaction times combined with reduced headway's and higher speeds were the cause of most of these accidents. This phase experienced the greatest reduction in accidents in the adapted-headway simulation. The only side effect of this strategy was a reduction in flows in high-density conditions [79].

A following-distance enforcement camera system used in the German state of North-Rhine Westphalia known as the Videoabstandsmessanlage (VAMA) resulted in a complete elimination of incidents of mass pile-ups and a reduction of accidents in general on the two test stretches where the VAMA plant have been installed [5, 8, 9].

This article presents a design for an alternative following-distance enforcement system that uses two X-Y scanning lasers per lane to detect tailgating vehicles. The first scanning laser would be able to detect towropes and connectors between vehicles such as a trailer, while the second would measure the speed of the vehicle. The gap would be measured by timing the period in-between vehicles. If an offending vehicle is detected the downstream bridge mounted video cameras would be triggered and the vehicle photographed by a flash-mounted camera downstream of the bridge. A speed measurement from the lasers would be used to time the operation of the camera. The camera would be placed only a short distance from the lasers to ensure accuracy. The video cameras could be mounted to a bridge or gantry, and observe the photographed vehicle for a pre-set time of say 30 seconds to be used as video evidence should the case go to court. The advantage this system would have over the VAMA system is that it would be fully automated and operate 24 hours a day.

2 The Effect of Accidents

The Autobahnpolizei-Koeln [7], the freeway police in the North-Rhine Westphalia (NRW) district of Germany, report that of the 20,000 traffic jams annually in NRW, on average, every second traffic jam results in an accident as approaching traffic hit stopped or braking vehicles. Due to high speed-differentials on freeways many of these accidents tend to be severe. US statistics report that 20% of all accidents are rear-end collisions, causing a significant number of fatalities and serious injuries. Strong impacts exceed human movement tolerance levels resulting in injuries such as whiplash or spinal injury [30, 39]. The Netherlands Police report following too-closely to be one of the top ten causes of accidents and actively target this practice through digital traffic surveillance technologies and in-car video evidence-based camera systems. They have found that 40% of all freeway accidents are caused by inadequate following-distances and that the following-distance allowance of 90% of drivers is inadequate [50, 51].

The German Federal Highways Institute (BaSt) reports that the incidence of pile-ups number around 300 per year, and account for 4% of all freeway deaths [13]. BaSt reports twice as many people are killed or injured in these accidents as in ordinary accidents. These accidents are typically centred around known blackspots. Fog is blamed as the cause of these accidents in six times as many cases as ordinary freeway accidents. The effect of trucks in rear-end accidents is substantial, with trucks involved in 50% of all accidents and 70% of all fatalities on freeways in Germany [13].

The Autobahnpolizei-Koeln [7], through their network of 3,500 traffic sensors built into the freeways of North-Rhine Westphalia (NRW) fed into a real-time traffic simulator have found an optimal high-density flow rate of about 80km/h. In such a state only slight disturbances are needed to destroy the equilibrium of flow. Strong braking by one car is enough to cause a traffic wave that will result in a jam several kilometres upstream. The brake impulse is magnified the further upstream it travels. They report over 50% of jams are caused by excessive volume of traffic, with about 25% caused by traffic accidents and less than 20% by road-work's [7].

US data claim that 60% of all *urban* traffic jams are caused by incidents [1]. Every minute spent clearing an accident tends to result in four to six minutes delay for drivers [46]. Barlovic [11] sees change in velocity as one of the causes of jamming. A velocity change within high-density flow can create a shock wave that passes upstream. Once the critical wavelength of this shock wave is exceeded, a jam will result [40, 78]. Staplin [55] defines the problem of high-density flow stating with flow increases the density of vehicles increases and the speed of vehicles decreases. The vehicle operators drive the speed and density of vehicles in this system. Drivers, he says will determine their following-distance and how fast they are willing to go. As these characteristics change with driver populations, he continues then the behaviour of the system and capacity will change.

Table 1: Rear-end accidents on freeways in Germany
(Data courtesy of Federal Highways Institute, Germany [14]).

Accidents on freeways	Injury accidents	Fatalities	Injuries	of which in fog		
				Injury accidents	Fatalities	injuries
1999 Rear end acc – stationary	1,038	40	1,816	6	0	14
Rear end acc – moving or waiting	10,052	260	17,588	24	1	43
Together	11,090	300	19,404	30	1	57
all accidents on freeways	26,627	911	41,910	53	1	99
2000 Rear end acc – stationary	1,015	36	1,732	4	0	11
Rear end acc – moving or waiting	9,573	243	16,646	31	1	120
Together	10,588	279	18,378	35	1	131
all accidents on freeways	25,578	907	40,198	58	1	168
2001 Rear end acc – stationary	1,007	50	1,860	2	2	18
Rear end acc – moving or waiting	9,729	183	16,983	16	1	36
Together	10,736	233	18,843	18	3	54
all accidents on freeways	25,990	770	41,069	43	6	105

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45/2002

Note: Stationary rear end accidents involve cars stopped on the shoulder, not in traffic.

Table 1 shows the incidence of rear-end accidents on freeways in Germany.

3 Pollution

The Highways Agency of the UK [60] conducted a study on the causes of pollution on freeways. They found that lower speeds resulted in the highest emission levels due to more stop/go driving and inefficient operation of engines at these speeds. There was an increase recorded once an efficient travel speed was reached, as table 2 shows.

Table 2 represents the problem of slow speeds in traffic flow. The engines of slower vehicles operate less efficiently, causing more emissions to be produced. As average speeds are increased to the optimum engine-speed engines begin to operate more efficiently. The emission of CO₂ is tabled for various speeds [60].

Speed variations caused by traffic waves in stop and go traffic can increase vehicle pollution emissions by up to 90%. Reductions in traffic speeds from 100km/h to 20km/h (due to jams) can increase emissions by about 50% [60].

Table 2: CO₂ Emissions – Light Duty Vehicles [60].

Average speed (km/h)	Emissions rate (g/km)
20	228
30	188
40	164
50	149
60	139
70	136
80	139
90	145
100	158
110	176
120	199

Data suggests that the optimum flow-state on a congested freeway occurs at speeds of 80km/h in the synchronous flow phase – see figure 1 [7]. Sudden braking or changes in speed can disturb the delicate equilibrium of this high-flow state resulting in magnification of the speed variation further upstream that could cause jamming [7]. It has been suggested that increasing the headway between vehicles could better protect flow stability and increase speeds, thereby decreasing fuel consumption and emissions [8, 11, 16, 47, 50, 55, 79].

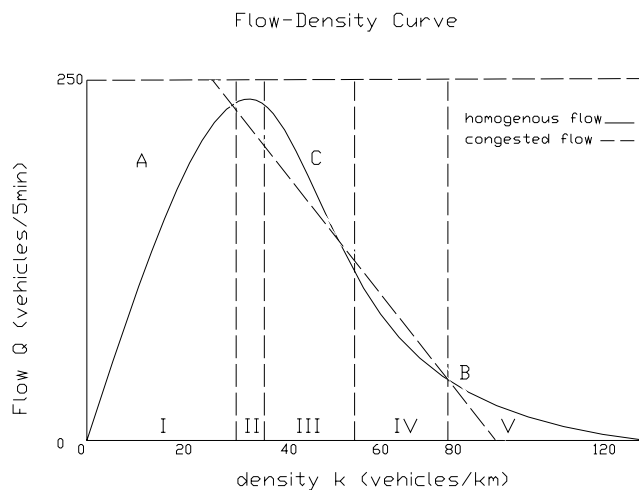


Figure 1: Q-k curve on the flow-density plane. A and B denote the free driving region and the jam points. C is the congested region [78].

Figure 1 illustrates the balance of flow and density. Region I is the free flowing region. Region II is the critical-flow region (synchronous flow) most sought after by traffic engineers to achieve maximum flows and roadway utilisation. Flow breaks-down rapidly with increasing density to region III, the congested flow state, leading into the most congested regions, IV and the stop-go region V [57, 78].

4 The Intelligent Driver Model

Martin Treiber [65] and his team at the Technical University of Dresden developed a downloadable simulator that models traffic flow dynamics. It is publicly available from <http://www.traffic-simulation.de>. Acceleration and deceleration are defined by the Intelligent Driver Model where acceleration dv/dt is dependent on the vehicles' velocity v , the distance to the lead vehicle s , and the velocity difference between the vehicles Δv (positive when approaching) [65]. v_0 is the desired velocity, a is the acceleration in everyday traffic and b is the braking threshold in everyday traffic, while T is the temporal following-distance. It assumes drivers adopt a tolerance for certain levels of a , b and T which influences their behaviour. Higher b or lower T values result in less anticipatory driving and greater wave generation. Low a values can have the same effect [66].

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

In dense traffic conditions corresponding to $v < v_0/2$ (1) the desired gap between vehicles s^* is determined by

- the minimum gap in bumper to bumper traffic s_0
- vT a velocity dependent contribution corresponding to a time headway T
- the velocity v
- the velocity differential between vehicles Δv
- the comfortable acceleration a and braking b values [63, 65].

$$s^* = s_0 + \left(vT + \frac{v\Delta v}{2\sqrt{ab}} \right). \quad (2)$$

The model assumes drivers will tend to drive in accordance with these thresholds; accelerating up to the desired gap s^* at their desired acceleration rate a , often failing to adequately anticipate the need to slow down in stop and go traffic. They will then be forced to brake at their desired braking threshold b . The driver's desired gap is dependent on their braking threshold – if a driver feels they can stop quickly then they tend to reduce their gap accordingly [63].

If the traffic density is very low (i.e. s is large) interaction between vehicles is negligible so the Intelligent Driver Model's acceleration is reduced to free-road acceleration where $a_f(v) = a(1 - v/v_0)^\delta$. The acceleration exponent δ defines how acceleration is reduced as the desired velocity is approached. The limited case $\delta \rightarrow \infty$ defines constant acceleration to the desired velocity v_0 while $\delta=1$ corresponds to an exponential relaxation to the desired velocity with a relaxation time of $t = v_0/a$. $\delta=4$ is proven to be a realistic value in simulations [67].

In emergency situations corresponding to $s < \Delta v^2 / 2b$ drivers will brake hard to avoid colliding with the lead vehicle [63]. This braking is shown as $b_{int}(s,v,v) = bk^2/b = \beta bk$. Where bk is the minimum kinematic deceleration needed to avoid a collision $bk := v^2/2s$, and the braking situation is said to be under control when $\beta := bk/b \leq 1$ [67].

A number of traffic scenarios are possible with this model. Truck ratios, traffic in-flows, speed limits, driver aggression and on-ramp volumes can be varied. A ring road, uphill grade and lane closure model are included. In contrast to the NaSch model, the IDM can simulate metastable states and hysteresis. Martin Treiber [66] advises that although the model has "been well-verified by experimental data" and "whole complex jam scenarios could be reduced" using it, "some features of human drivers are missing (no reaction time and no anticipation to next-to-next neighbours)". Treiber and Helbing [63] account for these shortcomings of the Intelligent Driver Model, reporting that although the Intelligent Driver Model fails to allow for a nominal driver's reaction time of 1 second this is offset by the driver's tendency to anticipate braking by scanning several cars ahead.

The model has been validated with empirical data obtained from a number of test sites on the German Autobahn network. These test sites were selected due to the prevalence of various types of high-density flow-states. Empirical data gathered from these sites were analysed. Historical and boundary data were fed into the simulation to replicate the real state. In one test an incident that blocked a lane of the study freeway was detected and cleared some time later. To replicate the effect of this incident the time headway T was increased from 1.6 sec's to 5 sec's and decreased linearly to 2.8 sec's during the 70 minutes that the incident lasted. This approach closely replicated the dynamics of an induced slow down. The outcome of these comparisons were density waves moving at -15km/h and closely simulated reproductions of the various congested states of flow that the empirical data recorded, including moving clusters, pinned local clusters and synchronous flows. The simulated phase diagrams reproduced the phase diagrams obtained from real traffic data, validating the model. Ramp flow effects were not tested [67].

5 Experiment

In testing the effects of headway with the Intelligent Driver Model this study shall be limited by a number of factors. The model is a scaled down version of the validated model, and as such the length of roadway, numbers of vehicles and hence its ability to reproduce high-definition flow dynamics is restricted. This study considers all drivers to be the same, as factoring in driver variation would be beyond the scope of this study. Accident-strength decelerations from driver's cutting-in or braking are difficult to simulate with this model, and therefore their effect will most likely be underestimated. Accidents are excluded, as this model cannot reproduce their effects on flow. Accident exclusion greatly limits the accuracy of the results that can be obtained. Since the effects of rear-end accidents comprise a major component of this study data obtained by others shall have to be relied-on. The effects of different weather conditions, such as rain and fog influence accidents and jams, but these effects are also beyond the scope of this study.

The Intelligent Driver Model (IDM) will be used to model the effects on flow of various headway treatments. The model uses the following quantified parameters - desired safety headway T in seconds, comfortable braking threshold b in m/s^2 , desired velocity v_0 in km/h , minimum distance to lead vehicle s_0 in metres, comfortable acceleration a in m/s^2 , acceleration exponent Δ and a politeness factor which is the lane bias - a low figure indicates a low lane bias and hence a greater propensity to change lanes.

Initial time headway's T shall be set to 1.0 seconds for cars and 1.0 seconds for trucks to reflect optimum conditions. Desired acceleration a shall be set to 0.5 m/s^2 to better allow for the slow to start rule - typically a is 1.0 m/s^2 . Braking threshold b shall be set to 4.0 m/s^2 , to simulate more aggressive braking as characterised by peak hour traffic - 3.0 m/s^2 is more typical of normal flow conditions. Inflow shall be set for the on-ramp and freeway boundary. Downstream flow in vehicles/hour shall be measured with a traffic detector that shall return the average speed and flow over the test period. This data shall be labeled the first control condition.

Changing T to 0.5 seconds for cars and T to 1 second for trucks the experiment shall be repeated, to produce a second alternative control condition, reflecting non-optimum conditions. This condition shall be labeled the second control condition.

Keeping all variables constant except the independent variable, time headway T the experiment shall be repeated. Time headway will be altered from 1 second to 2 seconds for cars and from 1 second to 2.5 seconds for trucks. Downstream flows and average velocities shall be recorded at the detector for the same test period as the control condition. Flow and average velocity shall be considered the dependent variables. This data shall be labeled the first test condition.

The experiment shall be repeated again, but this time the time headway shall be returned to the control condition. The braking threshold shall be decreased from 4 to 3 m/s² for cars and trucks in the IDM. Downstream flows and average velocities shall be recorded for the test period. This data shall be labelled the second test condition.

The three sets of data will be compared. If the flows and average speeds are unchanged in either of the test cases then the null hypothesis shall be considered supported for that test. If either of the test cases show a significant increase in flows or average speeds then the alternative hypothesis shall be considered supported – that is, increasing headway improves flow and/or speed. If there is a reduction in flows and/or speeds in either of the test cases then the amount of reduction shall be noted.

Table 3: T-Score comparisons of data.

t-Test SUMMARY - VELOCITY						
Velocity						
Control 1	Variable	t-Score	Significant	Control 2	t-Score	Significant
Cond1	Tcar=2.0	3,2309	Yes	Cond1	1,821144	No
Cond2	b=3.0	1,1160	No	Cond2	0,832509	No
Cond3	Tcar=3.0	3,9181	Yes	Cond3	1,433158	No
Cond4	Tcar=4.5	3,4657	Yes	Cond4	1,365012	No
Cond5	b=2.0	2,4256	Yes	Cond5	0,494478	No
Cond6	b=1.0	2,9645	Yes	Cond6	1,9156	No
t-Test SUMMARY - FLOW						
Flow						
Control 1	Variable	t-Score	Significant	Control 2	t-Score	Significant
Cond1	Tcar=2.0	-1,4082	No	Cond1	0,209599	No
Cond2	b=3.0	-1,0084	No	Cond2	0,694041	No
Cond3	Tcar=3.0	-3,0146	Yes	Cond3	-2,34739	Yes
Cond4	Tcar=4.5	-6,2481	Yes	Cond4	-3,4945	Yes
Cond5	b=2.0	-0,4196	No	Cond5	0,730782	No
Cond6	b=1.0	-0,3189	No	Cond6	0,757778	No
Control 2	Tcar=0.5	-2,3264	Yes			

Table 3 shows the t-scores and their significance using $p=0.05$, 1-tails with $df=9$ (i.e. 1.833) for every condition except conditions 3 to 6, where $df=4$ (i.e. 2.132).

Using $p=0.05$, 1-tails (i.e. 1.833) for every condition except conditions 3 to 6, (i.e. 2.132) the statistical analysis of the data produced t-scores and their significance as shown in table 3.

Using control 1 to represent the current traffic situation i.e. $T_{car}=1.0$, all the temporal headway conditions and conditions 5 and 6, $b=2.0$ and $b=1.0$, showed a significant increase in velocity. This was not the case in the second control, which had a higher standard deviation.

Significant reductions in flow occurred with respect to control 1 and control 2 in the higher temporal headway conditions 3 and 4. The alternative hypothesis that increasing headway will increase speed is supported by the experimental data, but the hypothesis that it will also increase flow is only partially supported. Flows did increase 9% from $T_{car}=0.5$ to $T_{car}=1.0$, possibly as a result of a reduction of flux and traffic waves, but this effect only applied up to $T_{car}=1.0$. The extreme settings of $a=0.5 \text{ m/s}^2$ - to better account for the slow to start rule and $b=4 \text{ m/s}^2$ - to better simulate more aggressive tailgating as typically occurs in peak hour traffic could have influenced this result.

Doubling the headway increased mean speeds by 50% in the first test condition, levelling off with higher headway's till speed plateaued at around $T_{car}=4.0$. Referring to the flow-density curve in figure 1 a similar relationship can be seen.

The positive effect of spreading platoons and breaking up clusters was visually observed during the simulation. Velocity increases of up to 50% testify to this (increasing headway's equate to increasing velocities [80]). Applying Robertson's platoon dispersion formula from Young [80] to the data, where

$$Q2(i+D) = F Q1(i) + (1-F) Q2(i+D-1) \quad (3)$$

and

$Q2(i)$ = the predicted downstream flow (vph) in the i 'th time interval

$Q1(i)$ = the flow of the initial platoon (vph) in the i 'th time interval

D = the average travel time over the distance for which the platoon dispersion is being calculated

F = the smoothing factor expressed as $1 / (1+0.5D)$ (4)

the table shown in table 4 was generated.

Table 4 shows the estimated platooning strength for each of the controls and test conditions. Test case 6, where $b=1.0$ showed the greatest platooning strength, while test case 4, where $T_{car}=4.5$ showed the least platooning strength.

Table 4: Platooning strength at the 300th second time interval.

	D	d	F	Q1	i	Q2	Strength
Control1	83.20429	66.56343	0.02917	3900	300	905.112	2
Control2	79.1174	63.29392	0.030631	3900	300	835.3129	6
Test1	55.40593	44.32474	0.043173	3900	300	884.7928	3
Test2	66.77177	53.41742	0.03609	3900	300	884.4559	4
Test3	52.05622	41.64498	0.045824	3900	300	761.9661	7
Test4	53.72974	42.98379	0.04446	3900	300	650.8101	8
Test5	62.47397	49.97918	0.038477	3900	300	882.6464	5
Test6	49.20318	39.36255	0.048353	3900	300	913.7962	1

6 Cost/Benefit of Treatment

On 1991 figures it has been estimated that each road-accident fatality costs the community in Australian dollars A\$962,115, each serious injury A\$67,874, each hospitalisation A\$51,139 and each minor injury accident A\$12,060 [62]. Haworth [29] estimates the average driver's value of time to be A\$11.33/hour. Comparing the costs and benefits of the preferred treatment regime to the control the following calculations need to be made. Although the effect of the treatment regime on rear-end accidents cannot be directly determined by this study the results of a similar study by Wanschura [79] can be referenced. Wanschura's [79] findings of a forty-fold reduction in accidents in a cellular automata simulation utilising an adaptive headway provide a means of comparison. Such a reduction in freeway accidents would reduce the annual Victorian (Australia) freeway rear-end accident rate with resultant savings to the community to that shown in table 5.

Table 5 compares the five year Victorian freeway rear-end accident statistics to the estimated post-treatment figures. Three different cases are presented -100% compliance with the treatment regime, 50% compliance and 25% compliance. Based on simulated accident reductions found in the Wanschura [79] adaptive headway study. Cost savings based on Torpey [62].

The optimum case assumes a 100% compliance rate with the treatment regime, which in reality would never happen. Altering the figures to a more realistic 50% or 25% compliance rate still should produce savings in the order of A\$41,000,000 over 5 years.

As each driver who leaves sufficient headway to absorb at least some of the energy of a traffic wave benefit all those that follow, even partial compliance with the treatment regime should result in significant reductions in wave propagation and accidents.

Table 5: Five year Victorian (Australia) rear-end freeway accidents, with estimated post-treatment comparisons.

YEAR	ACCIDENTS			TOTAL	KILLED	PEOPLE			UNINJURE	TOTAL
	FATAL	SERIOUS INJURY	OTHER INJURY			HOSPITALIS	OTHER INJURY			
TOTAL	5	224	1082	1311	7	285	1639	2920	4851	
\$COST					\$6,734,805	\$19,344,090	\$19,766,340		\$45,845,235	
TREATED	0	6	27	33	0	7	41	73	121	
SAVINGS	5	218	1055	1278	7	278	1598	2847	4730	
\$SAVING					\$6,734,805	\$18,868,972	\$19,271,880		\$44,875,657	
50%	0	11	54	65	0	14	82	146	242	
SAVINGS	5	213	1028	1246	7	271	1557	2774	4609	
\$SAVING					\$6,734,805	\$18,393,854	\$18,777,420		\$43,906,079	
25%	0	22	108	130	1	29	164	292	486	
SAVINGS	5	202	974	1181	6	256	1475	2628	4365	
\$SAVING					\$5,772,690	\$17,375,744	\$17,788,500		\$40,936,934	
	fatality	\$962,115	serious injury	\$67,874	hospitalised	\$51,139	other injury	\$12,060		

To get an appreciation for the hidden effect of rear-end accidents on flow some speculation and extrapolation of VicRoads [71] rear-end freeway accident data is needed. The 1,311 reported rear-end accidents on the 740km of Victorian freeways over five years equate to an annual rate of 0.35/km, or 0.001/km/day. Insurance companies are reluctant to provide data on the large number of unreported minor rear-end accidents, but it should not seem unreasonable to assume that around twice as many go unreported. This would produce an annual rate of 1.05/km, or 0.003/km/day. Over half the stretches of Victorian freeways are located in rural areas, with low traffic volumes and accident rates, so the over-representation of urban freeways in the data is a fair assumption. Peak traffic volumes on the Hume Freeway are in the order of 700vph with only 550vph on the Western Freeway [70], while the Westgate Freeway carries around 6,250vph on its four lanes [71]. Using the exposure method for accident analysis in Taylor [57] where the expected number of accidents at a given site $E(A)$ are a product of the exposure $f(q)$ and the accident propensity $g(q)$, the expected number of reported and unreported accidents for the 1km long, double lane simulated test stretch with 3,900vph peak flows was estimated to be 10 per year. The effect of downstream accidents on the test stretch cannot be determined, but an effect equal to additional 10 accidents per year has been assumed. Based on the findings in Wanschura [79] the number of accidents would be expected to drop by 19 per year. From Monroe's [46] lower estimate a delay of 4 minutes for every minute of clearing time with an annual effect on flow for the test stretch of 76 hours, or 13 minutes per day from accidents was arrived at.

The price of implementing the treatment regime is the congestion cost. Based on the experiment's findings flow reductions of between 5% and 8% for headway's over $T_{car}=2.0$ and braking thresholds less than $b=3.0$ can be expected. Flow reductions will increase congestion and delays. Wigan (1976) cited in Taylor [57] puts this cost as

$$C = (A + Bu + E/u^2)X + m \tag{5}$$

where X = the link length (km)

u = time taken per unit distance (hr's)

m = road tolls and direct road use costs (A\$)

A = operating cost of vehicle per unit distance (A\$0.60/km used [4])

B = traveller's valuation of travel time (A\$11.33/hr used [29])

The link travel cost is given by $C_{me} = \partial C_{te} / \partial q_e$ (6)

where C_{te} is the total travel cost on the link, given by $C_{te} = c_e q_e$ (7)

and q_e is the value of the congested flow.

According to Haworth [29] an average cost of travel time for urban arterial roads of A\$11.33/hr was used, based on 37.7% urban trips being business trips, 40.9% personal business/commuting and 21.4% leisure trips, with a \$30.05 hourly rate applied to business trips only.

Based on the experimental conditions with a link length of 1km (i.e. the detector's distance from the start of the virtual street) a generalised cost of travel for the different experimental conditions over the link was calculated as shown in table 6.

Table 6: Generalised cost of travel based on the 1st control.

Generalised Cost of Travel for the Link							C	Q	Cte	Annual Cost	
	A	B	u	X	E						
Control1	t=1.0	\$0.60	\$11.33	0.023	1	0	\$0.86	815.126	\$702.50		\$0.00
Control2	t=0.5	\$0.60	\$11.33	0.022	1	0	\$0.85	738.473	\$626.94	-\$1,111,539.63	
Test1	t=2.0	\$0.60	\$11.33	0.015	1	0	\$0.77	748.742	\$579.79	-\$962,629.60	
Test2	B=3	\$0.60	\$11.33	0.019	1	0	\$0.81	771.551	\$625.05	-\$631,877.94	
Test3	t=3.0	\$0.60	\$11.33	0.014	1	0	\$0.76	611.262	\$466.89	-\$2,956,217.19	
Test4	t=4.5	\$0.60	\$11.33	0.015	1	0	\$0.77	499.628	\$384.26	-\$4,575,013.78	
Test5	B=2.0	\$0.60	\$11.33	0.017	1	0	\$0.80	761.902	\$606.93	-\$771,797.39	
Test6	B=1.0	\$0.60	\$11.33	0.014	1	0	\$0.75	762.068	\$575.24	-\$769,390.24	

Table 6 compares the generalised cost of travel over the test link length (1 km) for each of the experimental conditions based on the first control $T=1$ sec. representing the current traffic conditions. The independent variable for each of the conditions is shown next to the test label. The cost imposed on each driver is shown in the bordered field under C, while the total cost per hour is shown in the Cte field. The annual congestion cost is shown in the last column.

Congestion costs were calculated by subtracting the outflow of each test condition from the control outflow, then multiplying this by the hourly travel cost of \$11.33 and the number of peak flow hours in a year, where bottleneck strength $\delta Q := Q_{ramp} + Q_{out} - Q_{out}'$ [68]. Peak flow hours do not normally exceed 6 hours per day, nor do they normally occur on weekends

or public holidays, so a peak period of 6*256 days was used. This approach produces high travel time costs, as it assumes that maximum flow reductions affect peak flow 6 hours a day, 256 days a year, but has the advantage of covering unforeseen costs such as freight delays that could be an issue on urban freeways but are difficult to price.

The treatment with the highest congestion cost of A\$4,575,014 was test condition 4, where $T_{car}=4.5$. Test condition 1, where $T_{car}=2.0$ produced a higher annual cost of A\$962,630 than test condition 2 at A\$631,878 where $b=3.0$, showing more anticipatory use of headway cost 34% less than a comparable fixed temporal headway regime. Larger differences favouring reduced b values were found with higher T values and lower b values, showing a 74% and 83% saving in test cases 3 versus 5 and 4 versus 6. The data shows control 1, with a headway of $T_{car}=1.0$ second and $b=4.0\text{m/s}^2$ incurred the least congestion cost. Control 1 produced the highest flows with high average speeds.

Additional to the aforementioned factors there is an additional cost/benefit for each treatment regime, and that is the environmental cost. Based on Haworth [28] the cost to society from car emissions is 0.11 Australian cents per kilometre. Applying the CO₂ emission rates provided by The Highways Agency UK [60] to the average speeds and hourly flows produced in each test condition a comparison between the environmental benefits/costs of each condition can be made. See table 7.

Table 7: CO₂ Emissions and environmental costs.

Emission Savings		Change in Speed							Rating	
Emissions rate (g/km)	Avg. Speed	-30	-20	-10	0	10	20	30 Avg.		
Control1	43.267	286.5771	214.932	180.1592	159.0995	145.733	138.0199	136.9801	180.2144	8
Cost\$		\$13,799.1184	\$10,349.2990	\$8,674.9364	\$7,660.8802	\$7,017.2631	\$6,645.8658	\$6,595.7978	\$8,677.5944	
Control2	45.502	267.1326	205.992	174.7952	155.747	143.498	137.3494	137.6506	174.594971	7
Cost\$		\$11,653.2382	\$8,986.0760	\$7,625.1648	\$6,794.2171	\$6,259.8738	\$5,991.6509	\$6,004.7902	\$7,616.4301	
Test1	64.975	176.06	156.5375	144.025	137.5075	137.4925	141.985	151.4675	149.296429	3
Cost\$		\$7,787.1404	\$6,923.6595	\$6,370.2311	\$6,081.9618	\$6,081.2984	\$6,280.0018	\$6,699.4132	\$6,603.3866	
Test2	53.915	212.34	178.604	158.1275	145.085	137.8255	137.1745	141.349	158.643643	6
Cost\$		\$9,677.9096	\$8,140.3097	\$7,207.0437	\$6,612.6001	\$6,281.7309	\$6,252.0600	\$6,442.3229	\$7,230.5681	
Test3	69.156	166.0256	150.266	139.844	136.2532	138.7468	144.4936	156.9028	147.504571	2
Cost\$		\$5,994.9776	\$5,425.9181	\$5,049.5927	\$4,919.9333	\$5,009.9741	\$5,217.4839	\$5,665.5647	\$5,326.2064	
Test4	67.002	195.1952	168.497	151.998	139.8994	135.1006	137.2012	141.1026	152.713429	1
Cost\$		\$5,761.0416	\$4,973.0640	\$4,486.1083	\$4,129.0271	\$3,987.3941	\$4,049.3917	\$4,164.5386	\$4,507.2236	
Test5	57.624	193.7024	167.564	151.376	139.7128	135.2872	137.5744	141.9112	152.446857	5
Cost\$		\$8,718.0474	\$7,541.6252	\$6,813.0449	\$6,288.1142	\$6,088.9293	\$6,191.8703	\$6,387.0585	\$6,861.2414	
Test6	73.166	159.251	145.834	138.0502	136.9498	140.8996	149.1158	163.6988	147.6856	4
Cost\$		\$7,169.0400	\$6,565.0437	\$6,214.6386	\$6,165.1016	\$6,342.9107	\$6,712.7813	\$7,369.2677	\$6,648.3977	

Table 7 applies the CO₂ emission rates provided by the Highways Agency of the UK [60] to the average speeds and hourly flows to determine the environmental cost to society of each treatment condition. Test 4, $T_{car}=4.5$, has the least cost with a 15% reduction in emissions

and a 47% reduction in environmental cost over the control. CO₂ emissions are in g/km, costs are A\$ annual/km.

Reduction in emissions over the control were found, with the greatest CO₂ g/km reduction of 18% found in test case 3, where T_{car}=3.0. The greatest reduction in environmental cost, of 47% was found in test case 4, where T_{car}=4.5, most likely due to the double effect of increased speeds and decreased flows. Test condition 1, T_{car}=2.0 produced a 17% reduction in emissions and a 10% reduction in costs while test condition 2, b=3.0 produced a 12% emission reduction and a 15% reduction in costs. Economic costs were based on annual flows and speeds, so reduction in flows produced a positive effect on cost savings, while emissions were based purely on average speeds. The latter was done to compare the pure benefits of the treatment regime to the control without including the negative side effect of flow reduction as a beneficial factor. Lifting vehicle speeds from inefficient low speeds to optimum engine running speeds is a significant factor in reducing emissions – see table 2 [60].

Figure 2 graphically compares the CO₂ g/km emissions to the congestion costs for each test condition.

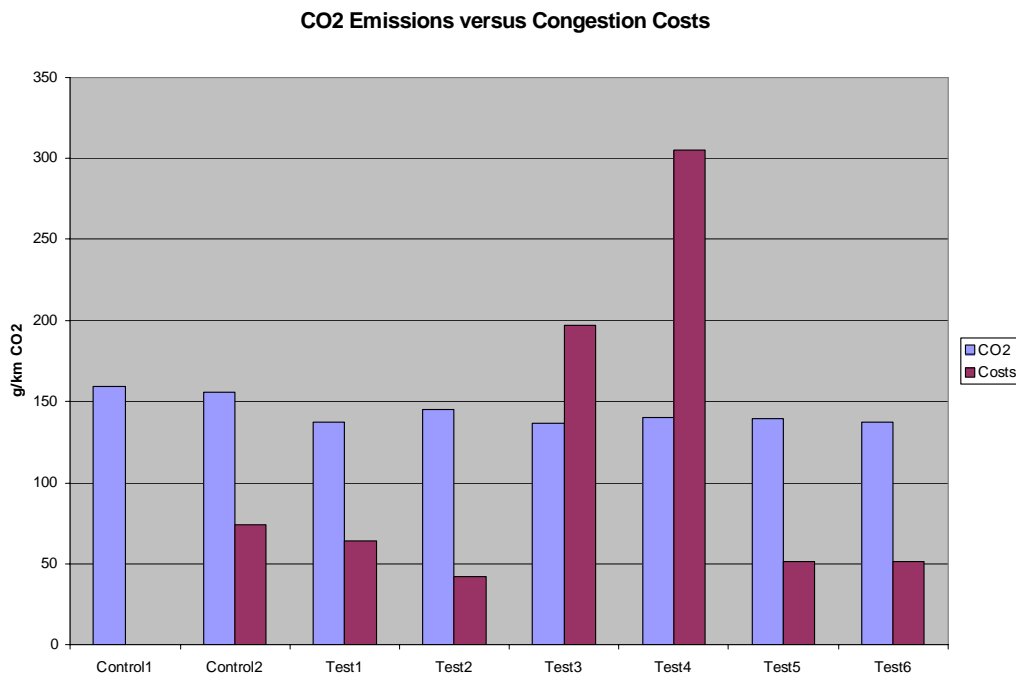


Figure 2: CO₂ Emissions and congestion costs.

Test condition 1, T_{car}=2.0 fared better than test condition 2 for CO₂ but not for congestion costs. Test condition 4, T_{car}=4.5 had one of the lowest CO₂ emissions but the highest conges-

tion costs while test condition 3, Tcar=3.0 had the lowest CO₂ g/km rate and the second highest congestion cost.

In comparing pure CO₂ g/km emissions to the congestion costs it can be seen that test condition 4 has the highest congestion cost while returning one of the lowest CO₂ emission rates. Test condition 3, Tcar=3.0 returned the lowest CO₂ emission rate of 147.5 g/km but had the second highest congestion cost. The first test condition, Tcar=2.0 fared better for CO₂ emissions than did the second test condition, b=3.0 although this reversed for congestion costs. Increasing headway seems to equate to CO₂ emission reductions.

Comparing the accident reduction savings to the congestion and environmental costs table 8 shows the different test conditions cost/benefits.

Table 8: Cost/Benefits based on the 1st control.

First Year Cost/Benefit Comparisons for the Test Stretch						Casualty		Net Benefit
	Capital Costs	Advertising	Congestion Costs	Accident Delay	Accident Reduction	Pollution		
Control1	Tcar=1.0	\$0.00	\$0.00	\$0.00	\$0.00	\$0	\$0.00	\$0.00
Control2	Tcar=0.5	\$0.00	\$0.00	\$1,111,539.63	\$0.00	-\$24,120	\$1,061.16	-\$1,134,598.47
Test1	Tcar=2.0	\$20,000.00	\$344.16	\$962,629.60	\$778,177.80	\$72,360	\$2,074.21	-\$130,361.75
Test2	b=3	\$20,000.00	\$344.16	\$631,877.94	\$778,177.80	\$72,360	\$1,447.03	\$199,762.73
Test3	Tcar=3.0	\$20,000.00	\$344.16	\$2,956,217.19	\$778,177.80	\$72,360	\$3,351.39	-\$2,122,672.15
Test4	Tcar=4.5	\$20,000.00	\$344.16	\$4,575,013.78	\$778,177.80	\$72,360	\$4,170.37	-\$3,740,649.77
Test5	b=2.0	\$20,000.00	\$344.16	\$771,797.39	\$778,177.80	\$72,360	\$1,816.35	\$60,212.61
Test6	b=1.0	\$20,000.00	\$344.16	\$769,390.24	\$778,177.80	\$72,360	\$2,029.20	\$62,832.60

Table 8 shows the net cost/benefit for the first year of each treatment regime based on the 1st control (Tcar =1 sec.) representing the current traffic conditions, comparing capital, advertising and congestion costs to the benefits of reductions in accident delay, casualty accidents and pollution.

Comparing the public education, congestion and capital costs of purchasing and installing the system on the test stretch to the benefits of reductions in accident delays, casualty accidents and pollution figure 3 rates the different test conditions. Increasing headway from Tcar=0.5 to Tcar=1.0 would produce a net benefit of A\$1,134,598. Test condition 2, where b=3.0 is the most cost effective treatment, with an annual net saving of A\$199,763 over the control condition, or A\$1,334,361 over control 2. Test condition 1, where Tcar=2.0, is estimated to produce a net annual cost of A\$130,362. Clearly a fixed headway is not the preferred treatment, with even higher net costs incurred by Tcar=3.0, of A\$2,122,672 and Tcar=4.5, of A\$3,740,650. To be cost effective, the system needs to be adaptive and dynamic.

Table 8 is presented graphically in figure 3. Figure 3 presents the first year of implementation cost/benefit for each test condition on the test stretch graphically. Based on the first control, Tcar=1.0 representing the current traffic situation.

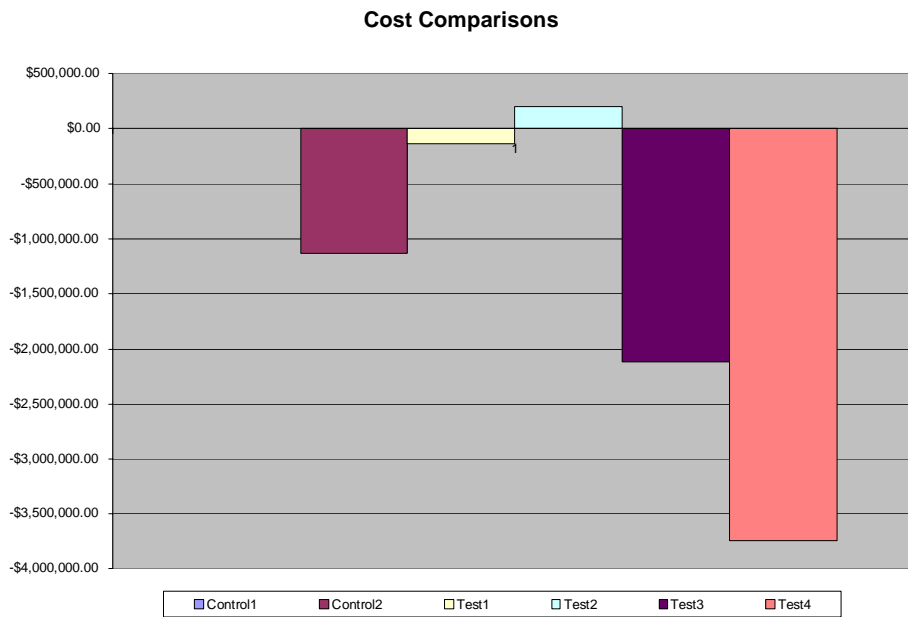


Figure 3: Cost/Benefits based on the 1st control.

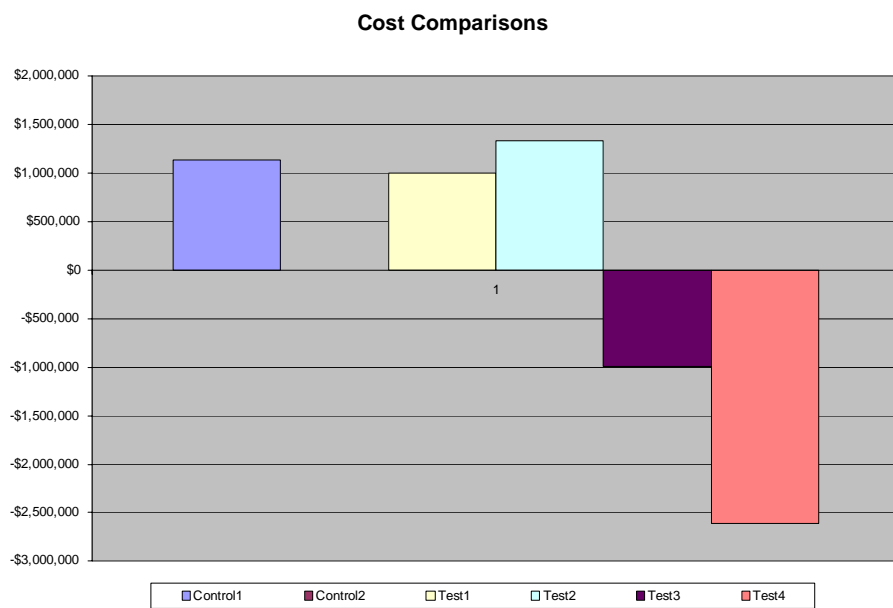


Figure 4: Cost/Benefits based on the 2nd control.

Figure 4 presents the first year of implementation cost/benefit for each test condition on the test stretch graphically. Based on the second control, $T_{car}=0.5$ representing the current traffic situation.

The cost/benefits in table 8 and figure 3 are based on control 1, with $T_{car}=1.0$ representing the current traffic condition. If the current traffic condition was better reflected by the second control, with $T_{car}=0.5$ and $T_{truck}=1.0$ then the annual cost/benefits for each test condition are far more positive, as shown in table 10.

Table 10: Cost/Benefit ratios based on the second control.

		First Year Cost/Benefit Comparisons for the Test Stretch				Casualty		
		Capital Costs	Advertising	Congestion Costs	Accident Delay	Accident Reduction	Pollution	Net Benefit
Control1	$T_{car}=1.0$	\$0.00	\$0.00	\$0.00	\$0.00	\$0	\$0.00	\$1,134,598
Control2	$T_{car}=0.5$	\$0.00	\$0.00	\$1,111,539.63	\$0.00	-\$24,120	\$1,061.16	\$0.00
Test1	$T_{car}=2.0$	\$20,000.00	\$344.16	\$962,629.60	\$778,177.80	\$72,360	\$2,074.21	\$1,004,236
Test2	$b=3$	\$20,000.00	\$344.16	\$631,877.94	\$778,177.80	\$72,360	\$1,447.03	\$1,334,361
Test3	$T_{car}=3.0$	\$20,000.00	\$344.16	\$2,956,217.19	\$778,177.80	\$72,360	\$3,351.39	-\$988,074
Test4	$T_{car}=4.5$	\$20,000.00	\$344.16	\$4,575,013.78	\$778,177.80	\$72,360	\$4,170.37	-\$2,606,052
Test5	$b=2.0$	\$20,000.00	\$344.16	\$771,797.39	\$778,177.80	\$72,360	\$1,816.35	\$60,212.61
Test6	$b=1.0$	\$20,000.00	\$344.16	\$769,390.24	\$778,177.80	\$72,360	\$2,029.20	\$62,832.60

From Haworth #104 Accident delay based on 6 hour peak period 256 days/year at 3900vph with a saving of 19 out of 20 accidents at a total daily delay for 20 accidents of 13 mins

Table 10 shows the net cost/benefit of each treatment regime, comparing capital, advertising and congestion costs to the benefits of reductions in accident delay, casualty accidents and pollution. Based on the second control ($T_{car}=0.5$ sec.) representing the current traffic situation.

If $T_{car}=0.5$ and $T_{truck}=1.0$ represented the current traffic situation the savings for implementing the second test condition $b=3.0$ would amount to a saving of A\$1,334,361 per annum. Net costs for the third and fourth test conditions would be reduced by A\$1,134,598 per annum.

As a worst case scenario, assuming that reported accidents are the only accidents to occur on the test stretch would produce an annual cost of A\$367,262 for test condition 2 over control 1, or a net annual benefit of A\$751,256 over control 2. In this scenario control 1 would produce a net annual benefit of A\$1,118,518 over control 2. These figures are based on 6 accidents per km per year, or only one every two months.

Due to the substantial congestion costs incurred by a vigorous treatment regime during high flow periods it would appear to be more economical to adjust headway warnings down to $T_{car}=1.0$ as per the first control condition during these periods to maximise flow. This approach would still reduce accidents by targeting high-risk drivers but would not greatly disturb flow. This would also be in accordance with the Dutch Video Control System practice of

using low headway values, $T_{car}=0.5$ as the trigger for infringements, extending the synchronous flow phase and reducing events that would lead to flow breakdown. Based on the positive data obtained when comparing control 1 to control 2 this would produce substantial cost savings.

In practice feeding the boundary conditions and flow data into a cellular automata simulator running the proposed following-distance warning system would allow the simulation to determine optimum flow conditions taking concentration and accident propensity into consideration, before returning the optimum headway T_{car} , T_{truck} settings to each sections' variable message sign (VMS) headway warning plant. A vehicle classification unit in the system would permit vehicle-type specific warnings. The feedback loop would be continually monitored and dynamically updated, constantly maximising the flow/headway relationship.

7 Treatment Solutions

Treatment of traffic waves and jamming through varying vehicle following-distance is a new field that is yet to gain wide spread acceptance. There are few systems that aim to study this effect in existence, while still others aimed at resolving the general problem of inadequate following-distances are still being trialed.

The Dutch Video Control System (VCS) is an example of the latter type of system. With 40% of freeway accidents in Holland caused by inadequate following-distance the Netherlands Police have targeted tailgating as one of their top-ten traffic infringements. Mounted on overpass bridges the VCS can oversee 400m of freeway, quickly assessing the following-distance of every vehicle passing through the control section. Vehicles with less than $\frac{1}{2}$ second (17m at the speed limit of 120km/h) following-distance are issued an infringement notice [51]. Portugal also uses a similar system [69].

A working example of a distance influence plant, the Videoabstandsmeßanlage (VAMA/VIBRAM) in Germany is a combination of the two types of systems. It aims to regulate the supply of new arrivals by spreading flow more evenly across sparse regions from dense regions while reducing rear-end accidents. Mounted on an overpass bridge the VAMA is comprised of two video cameras that give a multi-dimensional view of traffic movements within a range of up to 600m. The cameras measure the speed of accumulating vehicles and their following-distances as they cross over measuring lines. These lines are marked on the road surface as tabled in table 11 [8].

A third camera is placed on the median barrier and is used for vehicle identification. The average speed of vehicles is calculated from the measuring lines and the calibrated stop time is then faded onto the video.

Table 11: Measuring lines for the VAMA tailgating enforcement system.

	Line Length/width (m)
Zero point	0.3 x 0.3 (at height of the top of the bridge)
1. Measure Line 40 m from the zero point	1.5 x 0,5
2. Measure Line 90 m from the zero point	1.5 x 1,0
Position Line 340 m from the zero point	1.5 x 1,0

Enforcement is effected by means of two police officers in a police car equipped with a video-monitor located behind the bridge works. The video monitor has a live feed to the cameras on the bridge and can replay recorded videos of offences. An alternative asynchronous video-taped approach is also used, with processing done later in the office. Following-distance infringements are issued if the following-distance falls below the calculated safe reaction-distance, and if there have been no lane changes, brake applications or speed decreases within the 340m observation zone. Trucks and buses over 3.5 tonne are subject to special following-distance requirements and penalties [8].

The Transport, Road and Research Laboratory in the UK (TRRL) trialed an automatic following-distance warning sign (VMS) to test the effectiveness of a dynamic warning system. The number of tailgating drivers was reduced by a third because of the sign (Helliard-Symons and Ray, 1986 cited in Turnbull Fenner [69]).

This study is aware of the existence of a following-distance infringement system in use in Israel that uses three pavement reflectors targeted by three bridge mounted lasers supported by an enforcement camera, but has been unable to obtain reliable information on the system from the Israeli Department of Transport.

Chevron road markings accompanied by warning signs have been trialed in a number of countries including Australia. VicRoads trialed the system in 1999 at three sites. One of the sites (the Hume Freeway) showed a statistically significant positive effect, while the Western Freeway showed no significant effect. The Hume Freeway result only occurred in daytime, when drivers could see the chevrons. UK data showed a significant reduction in accidents over their trial period, although this could have been attributed to the stimulative effect that the chevrons had on driver fatigue. French data showed 40% of drivers did not understand the chevrons meaning. Canadian data showed drivers found the chevrons distracting [69].

A problem confronting efforts to increase the following-distance of vehicles on roads subject to congestion are findings that increasing the time headway of vehicles will result in decreases in flow. This reduction in flow could cause traffic jams [63]. A possible alternative to this is warning drivers of downstream traffic jams and incidents, thereby increasing anticipatory headway. Increasing driver's anticipatory behaviour by decreasing their comfortable braking thresholds has been shown in simulations to significantly decrease accidents, density waves and jams [63, 66, 79].

Reductions in the magnitude of sudden braking events and accidents could offset the negative effect of headway induced flow reductions. The balance is obviously a fine one, and the desirable following-distance would need to be carefully calculated for each particular traffic situation. Cars approaching a jam at speed would best be advised to increase their following-distance significantly as any braking manoeuvres undertaken in this situation would tend to be severe. Given the vulnerability of the high-volume synchronous flow phase to flow breakdown increasing following-distances to one second in this situation could well be beneficial [78].

8 Proposed Treatment System

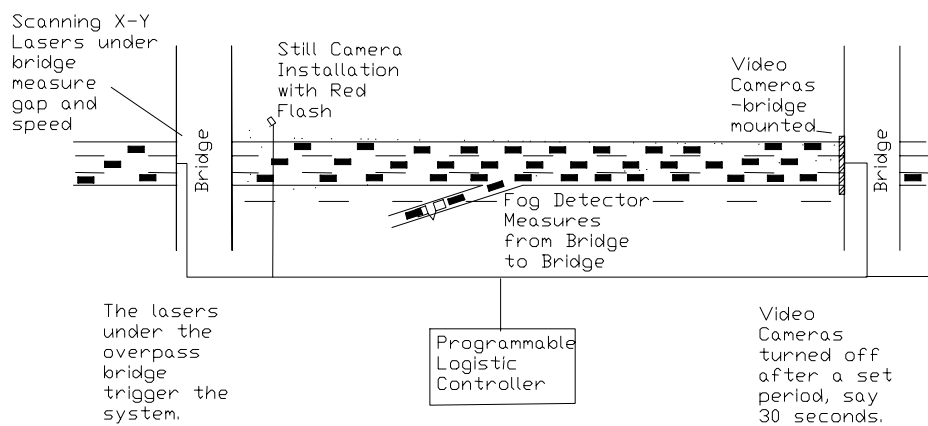


Figure 4: Layout of the proposed following-distance enforcement system.

Figure 4 shows the layout of the proposed following-distance enforcement system. Scanning X-Y lasers under a freeway bridge detect tailgating vehicles and trigger the still and video cameras. The still camera photographs the tailgating vehicle while the video camera on the next overpass bridge (or mounted on a gantry) videotapes the vehicle for a set period of time as evidence of the offence.

The proposed dynamic following-distance advisory system uses Variable Message Signs and a radar traffic detector to flash “TOO CLOSE” or “JAM AHEAD” warnings to drivers. Chevrons on the road allow drivers to judge their gap. Drivers ignoring this warning may be photographed by an downstream enforcement system. A problem, though, with such a system is driver confusion over to whom the warning message is directed [69]. As a remedy to this problem it is recommended that a white bar be painted on the road and accompanied by explanatory signs at the warning point. When the VMS flashes it's warning message the driver should be at the white bar. A speed measurement from the traffic sensors should be sufficient to achieve this co-ordination. Chevrons leading up to the bar accompanied by warning signs should help drivers get their headway right before reaching the sign.



Figure 5: Following-distance advisory system.

Figure 5 shows the proposed following-distance advisory system. The VMS on the gantry flash a warning to drivers depending on their speed and headway.

The proposed enforcement system uses X-Y scanning lasers mounted under an overpass bridge. Two beams per lane scan a reflective strip on the roadway. If the beam is broken the timer is set to standby. Once the beam rebounds off the reflective strip the timer starts, terminating once it is broken again. The second beam measures the velocity. In this way speed and following distance is taken.

Figure 6 shows the proposed following-distance enforcement system. Scanning X-Y lasers under a freeway bridge detect tailgating vehicles, while dismissing offences caused by vehicles connected together (such as towing).



Figure 6: Following-distance enforcement system.

Should a towrope or anything block the first beam no event will be triggered. Otherwise vehicles over 60kmh with a gap less than 0.5 second will be photographed by cameras on the departure side of the bridge using a red night-vision flash. A video and still camera installation is located on a pole some distance down the freeway from the bridge. When the lasers under the bridge detect a tailgating offence the camera installation commences video taping. The still camera is triggered from the speed-reading taken by the lasers.

An image intensifier on the camera allows the system to be used at nighttime. Image intensification would allow a clearer image to be obtained in poor visibility, as shown above. The system would offer 24/7 coverage without the need to be manned.



Figure 7: Following-distance enforcement system.

Figure 7 shows the application of image-intensification to allow tailgating vehicles to be videotaped in poor lighting conditions.

Table 13 shows the first year cost/benefit comparisons for the following-distance enforcement system, comparing accident, pollution and accident induced delay costs to the cost of flow-reductions from increased headway's and the capital cost to build the enforcement system.

Table 13: Cost/Benefit comparisons for the following-distance enforcement system.

Test2	Awareness			Casualty			Net Benefit
	Capital Costs	Advertising	Congestion Costs	Accident Delay	Accident Reduction	Pollution	
b=3 From Haworth #104	\$82,390.00	\$344.16	\$631,877.94	\$823,420.70	\$76,168	\$1,447.03	\$186,423.63
Travel Time Cost	Accident delay based on 6 hour peak period 256 days/year at 3900vph with a saving of 20 out of 20 rear-end accidents. at a total daily delay for 20 accidents of 13 mins						
37.7% urban trips business		\$30.05	\$30.05		\$11.33 lower		
40.9% personal business/commuting		\$0	\$12.18		\$18.92 upper		
21.4% leisure		\$0	\$12.18				

Table 14: Cost/Benefit comparisons for the following-distance advisory system.

Test2	Awareness			Casualty			Net Benefit
	Capital Costs	Advertising	Congestion Costs	Accident Delay	Accident Reduction	Pollution	
From Hawcb=3	\$29,700.00	\$344.16	\$631,877.94	\$778,177.80	\$72,360	\$1,447.03	\$190,062.73
Travel Time Cost	Accident delay based on 6 hour peak period 256 days/year at 3900vph with a saving of 19 out of 20 rear-end accidents.						
37.7% urban trips busine	at a total daily delay for 20 accidents of 13 mins						
40.9% personal business/commuting		\$30.05	\$30.05		\$11.33	lower	
21.4% leisure		\$0	\$12.18		\$18.92	upper	
		\$0	\$12.18				

Table 14 shows the first year cost/benefit comparisons for the following-distance advisory system, comparing accident, pollution and accident induced delay costs to the cost of flow-reductions from increased headway's and the capital cost to build the following-distance advisory system.

9 Summary and Outlook

It is predicted from previous studies using cellular automata traffic simulators' and European experience that accident reductions from increasing following-distances on busy freeways could be significant. Low compliance with following-distance safety advisories would still be expected to result in substantial reductions due to the flow-on effect that each safe driver has on the safety of drivers behind.

Evidence of reductions in traffic waves from increasing following-distances was not found by this study. Average speeds were increased by 50% when increasing the following-distance from 1 second to 2 seconds for cars and to 2 ½ seconds for trucks, but flow was reduced by 8%, with a negative net benefit. It was found increasing following-distance generally increased speed and decreased flow. Decreasing the braking threshold from 4m/s² to 3m/s² in the IDM produced a net benefit - taking accident reduction cost benefits into consideration, with average speeds increasing 25% and flow reducing 5%. CO₂ emission reductions were calculated at 12% per car over the untreated condition. CO₂ reductions occur when raising vehicle speeds to efficient engine running speeds.

To profit from the positive effect of using an adaptive following-distance as accomplished by reducing the braking threshold in the simulation it is suggested that Variable Message Signs (VMS) be used to warn tailgating drivers to increase their following-distances. Existing roadway flow detectors or VMS radar units could be used to detect tailgating vehicles, giving specific warnings to the offending driver. By integrating these signs into a controlled freeway individual following-distance warnings could be given, such as a situation-specific warning to increase following-distance should an accident or jam be detected, advising drivers closest to the incident to increase their following-distance the most, easing traffic into the slow-down

and preventing accidents. A fog detection system could warn drivers to increase their following-distance by a larger amount, the system knowing the location of the targeted vehicle in relation to the lead vehicle, which may be invisible to the targeted driver.

Following-distance warnings may be reduced or eliminated during high flow periods, targeting only drivers with less than a one second gap over 60km/h who are most likely to cause flow-disrupting accidents. Up to 25% of all freeway jams are caused by accidents. The synchronous peak-flow phase is the most efficient flow phase, but also has one of the shortest headway's and experiences the most accidents. Slight increases in the following-distances of dangerous drivers during these periods could result in substantial accident and accident-induced jamming reductions. Outside peak periods the benefits of the system would be expected to be entirely positive, due to lower flow volumes. This situation would make the adaptation of existing variable speed limit VMS to include following-distance warnings outside of peak periods more favourable, when the signs would not normally be required to display reduced speed limits.

Should compliance with following-distance warnings be low then a following-distance enforcement system could be employed downstream. Only a small number of these systems would be needed to ensure compliance.

Acknowledgements

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TABLE 1B. Wage Cost Index - Total Hourly Rates of Pay Excluding Bonuses, Australia, States and Territories (Quarterly Index Numbers)(a) (Australian Bureau of Sta-

- tistics,e-print www.abs.gov.au/ausstats/abs@.nsf/lookupresponses/2d5bde3973251273ca256aa8007d4529?opendocument).
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