Report

ITS Assisted Merging at Passing Lanes

Prepared for NZ Transport Agency (NZTA) (Client)

By Beca Infrastructure Ltd (Beca)

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Revision History

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1 Executive Summary

1.1 Introduction

This Report has been prepared for the New Zealand Transport Agency (NZTA) in accordance with Contract NO-09-510, “ITS Assisted Merging”. The report is on a literature review and microsimulation study into the use of intelligent transport systems (ITS), in particular variable message signing (VMS), to assist traffic merging at 2 to 1 lane reductions at the upper end of traffic volume for facilities such as (i) rural passing lanes (ii) rural 2+1 layouts and (iii) higher volume rural road sections planned for 4-laning, mainly peripheral to urban centres.

The work carried out is the first stage in assessing the technical and economic feasibility of ITS-assisted merging and comprises:

- (i) Preparation of a detailed offer of service (OOS) – 7 May 2009
- (ii) Literature review – in the OOS and progress report – 9 June 2009
- (iii) Development of test configurations and microsimulation analysis of operation
- (iv) Identification of operational sites and initial assessment of economic efficiency
- (v) Stage 1 Report to NZTA (this report), including ongoing research proposal for NZTA research programme funding

The ongoing research was envisaged to involve driver simulation to explore human factors in relation to the layout and form of VMS signage followed by on-road trials, possibly under controlled conditions depending on a safety assessment, and recommendation for operational deployment, should the results be favourable.

1.2 Literature Review

The literature review was based on sources already identified by NZTA and subsequent on-line searches. We did not locate any instances of ITS being used for merge assistance on rural passing lanes or 2+1 roads.

The practical uses of ITS for management of lane merging on rural highways appears to be limited to traffic control at highway construction zones, with all the examples being from North America. In most or all cases these were on divided highways and included 3 to 2, 3 to 1 and 2 to 1 merges and using portable ITS equipment or fixed signs. Concepts that have been employed are early merge, where traffic is encouraged to merge back into the main flow some way in advance of the lane end and late merge, where traffic is encouraged to make maximum use of both lanes up to the end of the lane. Both can be signed using static signs or variable messaging signs and have been called static early merge (SEM), static late merge (SLM), dynamic early merge (DEM) and dynamic late merge (DLM). There have been some limited field trials, not always successful and some compromised by equipment failures and other difficulties. Much of the published results have been from microsimulation studies some supplemented with field trials.

Reports of the capacity of 2 to 1 lane closures at work zones in the US are reported to be an average of 1340 veh/h one-way capacity over 7 studies. The field trials of DEM in the US have yielded inconclusive data, one concluding an increase in merge capacity of about 100 veh/h one-way compared with conventionally signed temporary traffic control, others have detected no difference and one study a slight reduction in capacity. No field results for DLM have been located; simulation has suggested a directional throughput capacity of 1730 pcu/h one-way for SLM and 1820 pcu/h one-way for DLM.

Evaluation of Swedish 2+1 roadways with cable barriers provides useful field verification of the upper limits of directional capacity of 2 to 1 rural two lane highway merges under conventional signing which was found to be in the range of 1,500 to 1,550 veh/h one-way based on a quarter hour peaking characteristic of 1,600 to 1,700 veh/h one-way. Other studies of 2+1 roads in other countries have generally indicated lower capacities in practice, down to 1200 veh/h one-way apart from one exceptional instance in Germany that demonstrated a directional capacity of 1,900 veh/h one-way.

Other examples of merge control using ITS are managed motorways in the UK and ramp metering, now used extensively in NZ. These examples do not provide any useful parallels for ITS merge signing on two lane highways.

Overall, the literature review does not provide examples of ITS-assisted merging for passing lanes on rural highways, and the research of ITS assisted merging has concentrated on temporary traffic control at highway work zones on multi-lane ,often divided, highways in North America. The evaluation of 2+1 layouts in Europe does provide practical examples of the capacity limits of these roads operating under fixed signage. At highway work zones the evidence both from field trials and from simulation is somewhat inconsistent as to the relative merits of DLM versus DEM strategies for 2 to 1 lane reductions.

1.3 VISSIM Microsimulation Modelling

1.3.1 Conceptual Layouts Tested

The main effort of the research was a microsimulation traffic modelling comparison of four basic scenarios: (i) passing lane closed; (ii) unassisted (conventionally signed) passing lane; (iii) VMS-message signing assisted early merging layout; and (iv) VMS-message signing assisted late merging layout.

The conceptual layouts for the ITS-assisted layouts were based on suggestions from NZTA as shown below and are discussed in detail in Section 4.

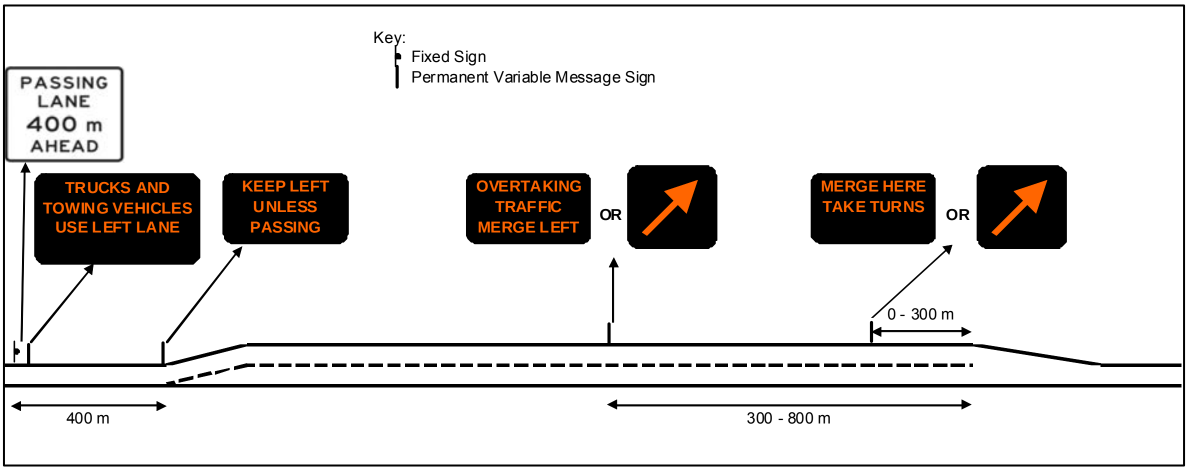


Figure 1 – Dynamic Early Merge, Diagrammatic Layout

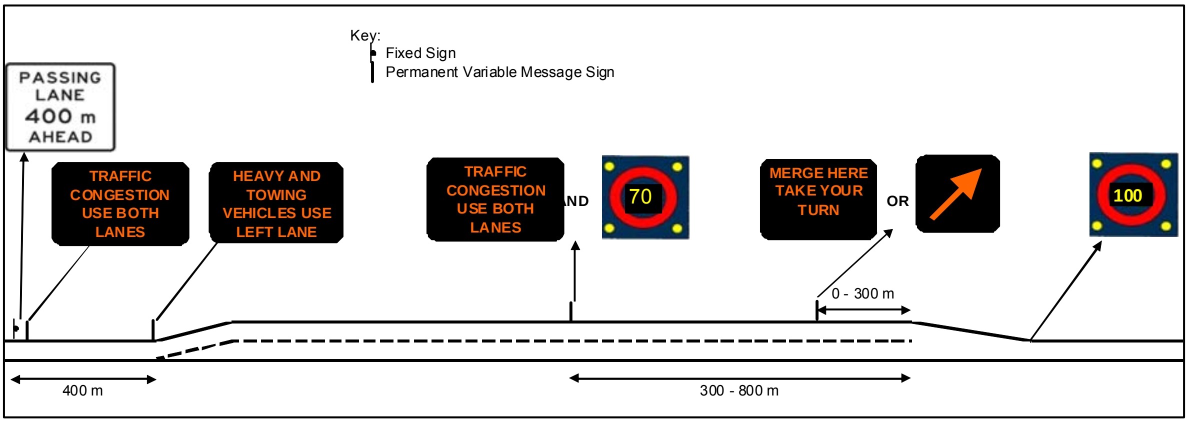


Figure 2 – Dynamic Late Merge, Diagrammatic Layout

1.3.2 Selection of Simulation Software

The suitability of several software packages were compared and VISSIM was selected as the most suitable for the context. The microsimulation software used for low to medium flow modelling of conventionally signed passing lanes such as TRARR does not have the features needed for VMS-assisted merging at high flow rates. The microsimulation packages PARAMICS, AIMSUN and VISSIM were compared, the latter found to be the easiest to use in this research context with the required adaptability, and VISSIM was eventually adopted.

1.3.3 Simulation Set-up and Options Tested

The simulation was for a straight flat road and single direction of flow as the software does not model overtaking in the opposing lanes. There was a lead-in length of single lane, which was varied between 5000 and 1000 metres, a passing lane varied between 1000 and 1500 metres, and a downstream length of 5000 metres. The layout represents passing lanes at about 5 km spacing. However, the downstream length of 5000 metres does not represent the 2+1 lane layout (continuous alternating passing lanes), which has a shorter downstream single lane length of 1500 to 2000 metres.

The increase in point mean traffic speed (and travel time saving) compared against a road without passing lanes, reduces with distance downstream of the merge point. US research of speeds downstream of passing lanes showed that the average travel speed is higher over the first 2.8 km (say 3 km) for all AADT ranges (Harwood, May, Anderson Leiman & Archilla, 1999. "Capacity and Quality of Service of Two-Lane Highways", Final Report 3-55(3) TRB cited in Koorey & Gu, 2001. "Assessing Passing Opportunities-Stage 3", Transfund Research Report No.22, 2001). For the modelled 1-1.5 km passing lane with 5 km downstream effective length, the last 2.2 km (i.e. about 44% of the downstream effective length) will therefore not show significant travel time savings. Therefore, the travel time savings for a continuous 2+1 layout will be greater than for passing lanes at 5 km spacing. Some further NZ field measurement on the size of speed savings at distances downstream of the merge over a range of traffic volumes and for passing lanes and 2+1 layouts would be useful.

The main outputs evaluated were the capacity of the merge (the flow rate immediately downstream of the merge point), the traffic speed immediately downstream of the merge and the travel time measured from 100m before the passing lane to the end of the simulation. As a variation on the DEM and DLM layouts, testing of NZ versus European lane markings at the merge point was also tested (slow merges into fast lane or fast merges into slow lane).

Two traffic streams were used composed of light and heavy vehicles, each with a distribution of desired speeds. The simulation software was adjusted to conform to NZ field research on vehicle speed distribution and platooned headway distribution. Changing the vehicle following sub-model calibration and, to a lesser extent, the lane changing calibration, was found to have a significant effect on the model performance (to the extent that using VISSIM default values would give considerably different results).

Each simulation option was replicated at least 5 times and the results averaged. Where the coefficient of variation of the output parameters was large, an additional 5 replications was made, giving 10 in all.

The simulation was run over a range of hourly directional traffic volumes from 250 up to 1800 veh/h one-way, for a base case of 1250m passing lane and 10% heavy vehicles. In the DLM option the lane selection was equalised and the base case was tested with the slower 50% (desired speed) using the left hand lane, with sensitivity to an equal division of light vehicles irrespective of desired speed between the lanes. In all cases a uniform flow rate was applied at the entry to the simulation over the 1 hour simulation (plus warm-up period). The vehicle stream at the entry to the simulation was randomly distributed, with the single lane length, the desired speed distribution and the traffic volume influencing the degree of platooning that had built up ahead of the passing section.

Sensitivity tests were then made at a demand flow of 1500 veh/h separately (i) for percentage of heavy vehicles (10%, 20%), (ii) for passing lane length (1500, 1250, 1000m), (iii) for positioning of the VMS sign for early merge, (iv) for including speed restriction for late merge (100km/h, 70 km/h, 50 km/h). Some supplementary sensitivity tests were made at 500 veh/h one-way to check behaviour of the layout at a lower demand flow.

1.3.4 Simulation Results

The leading results from the simulation were:

- The conventionally signed (unassisted) passing lane was used as the base for comparison of the performance of the other layouts (closed PL, DEM, DLM). At 10% HCV, the throughput of the unassisted PL was about 1,350 veh/h one-way compared with 1,550 veh/h one-way for ITS assisted merging at the end of the passing lane, a difference of 200 veh/h. However, based on information provided by NZTA, passing lanes are generally closed at about 1,200 veh/h one-way to allow for traffic flow variations. In practice, the current upper limit for unassisted merging is 1,200 veh/h and the difference is 350 veh/h one-way rather than a difference of 200 veh/h one-way The graph of demand volume against throughput capacity was almost identical for the DEM as for the passing lane closed, an advantage of 350 veh/h over the unassisted passing lane, while the throughput capacity for the DLM was 1,520 veh/h, an improvement of 320 veh/h one-way.

- In terms of traffic speed immediately downstream of the passing section, the speed for the unassisted passing lane dropped to under 70 km/h, compared to 80 km/h for the passing lane closed, 77 km/h for the DEM and 75 km/h for the DLM

- Travel time from the start of the passing section to the end of the simulation is the critical output for evaluating the economics of the ITS options. Based on a 5 km downstream length, at 1,500 veh/h one-way demand flow, the time savings of the DEM layout relative to the unassisted passing lane averaged 0.91 minute/vehicle compared with 0.78 minutes for the PL closed and 0.61 minutes for the DLM. The DEM exhibited a slightly higher saving than the closed PL, attributable to having been able to accomplish some vehicle passing and achieve slightly higher speeds over the PL section.

- The operating merge capacity results were sensitive to the percentage of HCVs (and by inference slow vehicles). Increasing the proportion of HCVs from 10% to 20% increased the differences between the unassisted PL and the other options
- Varying the length of the passing lanes between 1500 m and 1000 m had no significant effect on the throughput, downstream speed or travel time saving of the options
- Varying the length of single lane section before the passing lane (from 5000 m to 1000 m) had only a small effect on the simulation outputs at a demand flow of 1500 veh/h one-way. This was because the flow was heavily platooned in all cases after a short distance from the start of the simulation.
- The positioning of the merge signing for the DEM was varied between 750 m, 500 m and 250 m of the end of the PL. There was a deterioration of performance between 500 m and 250 m but none between 500 m and 750 m
- A comparison of the NZ and European merge markings showed a slight capacity advantage for the DEM but none was distinguishable for DLM.
- Adding speed restrictions to the DLM of 70 km/h and 50 km/h showed a slight decrease (10%) in travel time saving compared with the unassisted PL at 50 km/h but no decrease in travel time savings at 70 km/h, although there was a small speed reduction.
- The DLM simulation assumed that slower vehicles kept to the left lane; a sensitivity test that kept heavy vehicles to the left but otherwise allowed traffic to distribute between the lanes irrespective of the their desired speed gave a 20% reduction in the travel time saving of the DLM compared with an unassisted passing lane. This reduction would suggest that for DLM, there was little gain in having traffic with mixed desired speed travel in both lanes without passing.

1.3.5 Discussion of the Simulation Results

The microsimulation showed the ITS-assisted early merge (DEM) to have significant operational advantages in regard to throughput capacity and travel time over an unassisted passing lane. The ITS-assisted late merge (DLM) also showed advantages but less than for the DEM.

There was no evidence of the anticipated superiority of a DEM layout for intermediate flow rates transferring to an advantage for a DLM arrangement for heavier flow rates. On the basis of the simulation, the only advantage of a late merge is the ability to hold slightly more queuing traffic within the two lane section compared with an early merge. It is not possible to compare queue lengths between the options as the formation of a queue is a transient effect and occurs only when the demand flow exceeds the capacity of the merge.

The difference in capacity flow rate between DLM and an unassisted PL is modelled at about 130 veh/h. The queuing capacity per 1,000m of traffic lane, assuming a speed of 70 km/h, is about 35 vehicles, or a little less than 15 minutes of the excess flow. The implication is that once flows approach or exceed the throughput of the merge, queues will build up quickly and a 1.0 to 1.5km passing lane will provide relatively little space to accommodate excess vehicles.

The simulation was sensitive to some of vehicle following and lane change parameters. At the default values in VISSIM, the capacity of the DEM and DLM were both slightly less than with the calibrated model. We are conscious that the simulation assumes rational driver behaviour but does not allow for traffic incidents and extremes of behaviour. Also, in order to avoid vehicles becoming “stuck” at the merge point under high flow, which can occur in VISSIM and these vehicles are eventually removed from the simulation. To avoid this the lane switching parameters were set quite aggressively, implying a high level of driver responsiveness. While tests did not show this to have significant effects on the results, in a practical situation drivers may not respond to the level modelled, in which case we would expect field trials to demonstrate slightly lower capacities and travel time advantage.

Applying a speed restriction to the layout started to detract from performance when brought down to 50 km/h, but had little effect at 70 km/h. However, there may be safety issues with the high flow cases that would benefit from the calming effect of a VMS speed sign when traffic levels are high.

In regard to the advance distance of a VMS merge sign under DEM, 500m appears to be a suitable compromise between obtaining the maximum passing length and achieving sufficient gaps for a smooth merge. However, this is a parameter that could benefit from testing in field trials.

1.4 Potential Operational Advantages of ITS-Assisted Merging

Passing lanes are currently closed at flows above about 1,200 veh/h one-way. Consequently, any extension of passing lane operability above this flow rate will provide benefits in travel time savings. Between 1,200 and 1,350 veh/h one way, the travel time savings of DEM compared with passing lane closure appear the same as for a conventional passing lane (unassisted with ITS). Between 1,350 and 1,500 veh/h one way, DEM appears to provide additional travel time savings compared with an unassisted passing lane, as well as with the passing lane closed. The optimum ITS layout will require further stages of research into driver awareness, comprehension and response to different physical layouts.

A question to be resolved is how to close the passing lane if one-way flows exceed 1,500 veh/h and reach or exceed the merge capacity of the ITS-assisted layout. After discussion with NZTA, it was suggested that for costing purposes the range of physical layouts considered could be widened to include a less complex arrangement than that originally proposed by NZTA in Figure 1 and Figure 2 and this was subsequently included as an additional option in the economic analysis.

The simplified arrangement would dispense with a DLM option as the simulation shows it to be of marginal if any effect, and would use the remaining VMS signage to close the passing lane at traffic flows exceeding 1,500 veh/h (one-way). The VMS sign 400m in advance of the passing lane, at the passing lane start and at about 500 m before the merge point would be changed progressively to read “passing lane closed, use right hand lane” (or similar message such as “straddle lanes”), possibly with an enforcement camera linked to induction loops within the carriageway and central shoulders to help deter overtaking. If DLM were not included, then the VMS sign at the merge point could be dispensed with, retaining the normal fixed signage at this point. An alternative to directing traffic to use the right hand lane could be to require traffic to straddle both lanes. However, further investigation of these ideas is required.

1.5 Economic Benefits and Costs

The costs of a VMS installation will depend somewhat on location and whether a single side, gated layout or overhead gantry system is required.

Based on a DEM/DLM layout as in Figure 1 and Figure 2, assuming signage on both sides of the road (gated layout) within the passing lane section, we have roughly estimated a discounted cost per passing lane of $0.7 million for a DEM/DLM layout, inclusive of $0.5 million capital cost, 5% annual O&M costs and 15 year replacement cycle, compared with a conventional passing lane. The simplified layout providing DEM with VMS signs to control passing lane closure suggested by NZTA is also estimated at $0.70 million. A gantry arrangement would be comparatively higher cost and has not been included in the economic analysis.

The cost savings and benefits of an ITS-assisted merge installation will potentially comprise: (i) cost savings from deferring four-laning of road sections that are approaching capacity; (ii) reduction in travel time and congestion between the DEM installation and passing lane closure over the 1,200 to 1,500 veh/h one-way flow range; and (iii) operational savings by not having to manually close the passing lanes.

A rough order of costs and benefits analysis has been made, and this indicates that a simplified DEM installation as suggested by NZTA, under a wide range of input assumptions regarding capital costs, hourly traffic flow profile over the year and traffic growth rate, would pay for itself in cost savings to the road authority over the 15 year evaluation period through deferred construction and avoided operating costs of manual closures of passing lanes during peak periods where directional peak flow exceeds 1,200 veh/h.

Circumstances favourable to the economics of a DEM installation are:

- Relatively high cost of land and construction for adding or extending an auxiliary lane, such as in difficult terrain and geology and/or where land values are high on the urban periphery;
- Traffic flow at or approaching 1200 veh/h one-way for 100 -150 hours per year, in the region of 15,000 vehicles/day, depending on the shape of the annual hourly flow profile;
- Relatively low traffic growth rate, which has the effect on increasing the time that four-laning can be deferred by ITS-management of passing lane capacity;

Conversely, a DEM installation will not be as economically beneficial where the land and construction costs are relatively low, and/or peak traffic flows are still below 1200 veh/h one-way, and/or traffic growth rates are relatively high. A high traffic growth rate will tend to limit the number of years where the ITS-managed extension to passing lance capacity is effective before four-laning becomes the better option.

The rough order of cost and benefit analysis indicates that there is a reasonable prospect of permanent ITS installations located on selected road sections giving sufficient benefit to justify their expenditure. However, it would be necessary to carry out preliminary design and costing for specimen trial installations to confirm this in any particular case. This should be done following further examination of driver comprehension and reaction to trial layouts based on a driver traffic simulator to establish the best positioning and form of VMS messages.

1.6 Potential Market for ITS-Assisted Merging

NZTA has estimated that over the next 30 years there will be about 900 km of rural state highway, mainly in flat and rolling terrain, which will move from under 10,000 veh/day to between 10,000 and 25,000 veh/day, demanding more than a two-lane road with periodic passing lanes. There is an additional 200 km of rural state highway already within that flow range but not part of proposed four-laning.

A full 2+1 layout would potentially defer the need for four-laning, particularly if the upper limit of operation of 2+1 could be extended from about 17,000-21,000 vpd (i.e. 1,200 veh/h peak one-way flow, 60/40 - 55/45 directional split and respectively 12-10.5% AADT peak hour flow) up to about 21,000-26,000 vpd (i.e. 1,500-1,800 veh/h peak one-way flow). At these AADTs, the passing lane would be closed for about 100-150 (say 125) hours within the final year of the analysis period. At a typically high 2.5 % p.a. growth rate, the 4,000-5,000 vpd difference is equivalent to respectively about 9-12 years extra traffic growth in addition to the 22-36 year operating life of the facility. The 22-36 year service life assumes a 2.5 % p.a. growth rate between 11,000 vpd and respectively 17,000-21,000 vpd. This extended operating life would be particularly beneficial if constructing 2+1 layouts on commuter routes, as ITS-assisted merging could extend the operating life out to about

30 years which is similar to the project analysis period under NZTA's Economic Evaluation Manual procedures.

In addition there may also be several isolated passing lanes where seasonal peaks may warrant either a permanent or temporary ITS treatment.

1.7 Conclusions and Next Steps

Overall, the results of the microsimulation and the evidence from overseas microsimulation and field research, together with the indicative comparison of benefits and costs, together provide sufficient justification in our view for further research and development of the ITS-assisted merge concept applied to passing lanes and 2+1 road layouts operating at flow rates approaching capacity.

The next steps proposed are:

- Stage 2 – driving simulator based research to investigate the driver response to ITS layouts, in terms of driver perception, comprehension and behavioural response to different ITS signing arrangements; the outcome of this stage would be confirmation or otherwise of the driver responses, particularly at the merge points, and selection of a preferred arrangement of signing; development of a specimen design and costing, and a report suitable for taking the project forward into Stage 3, on-road trials. The indicative budget for Stage 2 is estimated at $100,000 and a 12 month time period.
- Stage 3 – road trials, which may be one or two stage, the first stage could be carried out off the public road to be satisfied with general feasibility of running the trial and with road safety; followed by an on-road trial at a selected site after discussion with NZTA.. ITS-assisted merging could possibly be undertaken on a trial basis either at an existing passing lane with current merge problems or a merge could be provided in one direction on an existing four lane carriageway using cones with truck mounted VMS messages. Costs and time frame for Stage 3 would be determined at the end of Stage 2.

2 Background

2.1 Rationale and Objective

New Zealand's main rural road network is, with limited exceptions, made up of two-lane highways on a variety of terrain from flat straight alignment to tortuous alignment in mountainous terrain. As traffic volumes rise, the capacity of the rural network to operate at an acceptable level of service becomes constrained and passing lanes are used as an intermediate solution to capacity enhancement, short of the more costly solution of four-laning. The logical next step after passing lanes in series is a 2+1 alternating passing lane configuration with a central median cable or gap separation, as has been extensively used in Scandinavia and is now being considered for deployment in New Zealand.

The effectiveness of passing lanes is governed by their length, platooning of vehicles entering the passing lane section and the speed differential between vehicles in the traffic stream. This tends to be greatest on upgrades and on hilly to mountainous alignment. The operation of passing lanes relies on judgements made by the motorist on the available residual passing length, forward visibility, the speed of overtaken vehicles, the behaviour of overtaking vehicles immediately ahead and the perceived risks from oncoming traffic.

Increasing the range of traffic volumes over which passing lanes operate efficiently is one such application. At present, if the directional traffic volume on a two lane road is high, such as during a holiday weekend, then it is common practice to close the passing lane at about 1200 veh/h one-way to avoid safety problems at the merge and to maintain road capacity. In such conditions it has been found that a passing lane can lead to a net reduction in capacity due to congestion at the merge point – the stable flow regime is disrupted by vehicles negotiating the merge at the end of the lane sending shock waves back along the traffic stream.

In line with the NZTS objective of making the most effective use of existing transport infrastructure, the aim of the research is to investigate the extent to which intelligent transport systems (ITS), can be used to modify driver behaviour, and reduce road user and road provider costs.

The potential benefits stem from (a) increased practical capacity of existing 2 to 1 lane reductions before flow instability and jam conditions set in and/or (b) deferring the need to construct additional lanes.

Locations where these conditions are met could include (i) existing rural passing lanes in series operating at high volumes where a higher level of infrastructure is still not justified for the majority of the year, (ii) 2+1 layouts on higher volume rural road sections; (iii) rural road sections earmarked for 4-laning, mainly near the urban periphery where a three-lane arrangement may defer or avert the need for 4-laning.

2.2 Key Outputs

NZTA's primary objectives of the research are:

- To identify and recommend a preferred layout of signs and messages;
- To determining the minimum and desirable length of the two-lane section (passing lane or 2+1) for an adequate level of service;
- To determining the likely tailback effects when operating under assisted merging, in relation to the 2-lane length;

- To identify the operating capacity for the recommended sign layout and minimum 2- lane length and under what traffic conditioned the standard signs layout should transition to ITS-assisted merging (early and/or late)

2.3 Overseas Examples

NZTA has identified theoretical and computer simulation-based research carried out in the USA into the use of ITS assisted merges at construction work zones on multi-lane highways. While static signing of closures works satisfactorily at low levels of traffic flow, as volumes rise there is a tendency for motorists to form a queue in the open lane in advance of the merge point, while opportunistic and uninformed drivers continue in the closing lane and force re-entry to the traffic stream to the frustration of other motorists. If traffic could be encouraged or controlled to a more orderly "merge like a zip", then progress in both lanes to the merge point would be recognised as legitimate behaviour, optimal use of the roadspace up to the end of the passing lane would be gained, and these sites would potentially be able to operate under stable flow conditions up to a higher traffic volume.

2.4 Assisted Merge Control Strategies

Strategies for assisting more orderly and efficient merge control vary from simple measures such as new forms of static signs which indicate the expected queue behaviour, through to more sophisticated forms of active merge control, varying with traffic conditions and implemented using variable traffic signing (VTS), one form of ITS.

Ideally, drivers would be encouraged to move from the behaviour that operates at normal flow volumes, where overtaking vehicles remerge at high speed towards the end of the passing lane, to a merge behaviour appropriate at heavy traffic volumes where the speed of traffic will tend to be lower, and merging is guided or controlled by rules and signage that become active only when the heavy traffic conditions occur.

There are two recognised forms of these assisted merge strategies, the early merge and late merge, which have been experimented with at road construction zones:

- Early merge – traffic is encouraged to merge by advisory signs signalling the lane drop (the so-called "static" form of merge assistance) or by a "dynamic" form where some form of variable warning is given that encourages or requires merging (by no passing signs) well in advance of the lane drop.

- Late merge – traffic is encouraged to use both lanes until close to the merge point and then required to "take your turn" merge; again a static form using fixed signing, and a dynamic form, which was deployed only in the higher congestion periods, using VMS linked to traffic detectors

The findings of Beacher et al (2004) at highway work zones in Virginia USA indicates that there are no unequivocal conclusions about the comparative efficacy of these merging strategies for 2 to 1 merges (3 to 1 merges show clearer advantages but are not relevant to the passing lane case). The proportion of heavy vehicles appears to have an effect on the performance of the various assisted merge strategies, efficacy increasing with proportion of heavy commercial vehicles, and this can possibly be extended to "slow vehicles" in general. The types of signs, conspicuity and wording all appear to have an influence on performance.

2.5 Adaptation to Rural 2 to 1 Lane Reductions in New Zealand

Adapting assisted merge strategies to rural 2 to 1 lane reductions in New Zealand involves a translation of both country and traffic context.

Traffic Context: Traffic control at work zones on multi-lane highways is a different traffic context from fixed passing lanes operating under normal operational conditions. Work zone traffic control is one facet of ITS-assisted management of traffic on high volume roads, at the upper end of which are dynamic motorway traffic management systems such as are being trialled in the UK.

In Europe, Sweden in particular, 2+1 road layouts have been extensively used as an alternative to their generous two-lane layouts (3.5 traffic lane + 3.0m paved shoulder). These are starting to be deployed in New Zealand.

Country Differences: arise from (i) traffic regulations, enforcement and penalties, (ii) levels of driver education and experience, (iii) the habits and psychology of the driving population including the general degree of observance of road rules, (iv) general national or regional behavioural characteristics and (iv) differences in the vehicle fleet. These human, legal and technical factors mean that research and experimental findings in one country will not necessarily translate to another without some modification.

2.6 Research Issues

2.6.1 The Driver Behaviour Objective

The ultimate objective is to influence driver behaviour at merges in traffic streams approaching capacity. If all vehicles could be brought under remote control, then a suitable algorithm should be able to arrange speed and merge trajectories to avoid creation of flow instability up to the point of maximum capacity. As this is not currently a practical option, the aim is to find a signing strategy employing a mix of fixed and variable, message and warning, signs that will optimise driver behaviour at reasonable cost, give worthwhile road user savings and not compromise traffic safety.

2.6.2 Defining the Potential Market and Economic Justification

It is useful to define the size of the market and the potential total benefit that might be achieved in order to justify implementing ITS-assisted merging. This involves identifying (i) the upper limit of traffic volume at which rural 2 to 1 lane reductions cease to operate effectively (ii) the number of potential sites and (iii) the hours/year of operation and total volume of traffic that stand to be affected. The research should also keep in mind that a more effective merge control strategy could potentially have wider applications than rural 2 to 1 lane reductions, including temporary traffic control at work zones and traffic incident sites, and at lane drops on expressway and motorway systems

In economic terms, the reduction in road user costs over time (time, congestion, VOC and crash costs) should exceed the cost of providing and operating the signage net of any savings in deferred road upgrading (lengthening or adding passing lanes or four-laning). Knowing the potential size of the market and the economic return are decision factors on whether to proceed with later stages of a research programme.

It is also likely that a dynamic management strategy using ITS will cost more than a static passive strategy involving only fixed signing, and so the benefits will need to be proportionately larger and might apply in different traffic situations.

2.6.3 Understanding the Traffic Dynamics

An analysis of the dynamics of assisted traffic merges is needed to design potential layouts for the signing of the ITS assisted merge strategies. A microsimulation testbed appeared the obvious way of approaching this, using one of the available software platforms of AIMSUN, VISSIM or PARAMICS. The simulation packages designed for modelling passing behaviour on two-lane roads, TWOPASS and TRARR, unfortunately do not have the inbuilt features needed to model the ITS strategies.

2.6.4 Defining the Message and Delivery

The research needs to be clear about the messages that need to be delivered to the driver. This could be either a combination of signs at different points along the lane or a very limited number of signs at specific locations. However where possible some rationalisation of messages would be required to simplify the merge process for drivers. The possible messages are:

- No passing/merge left
- Merge like a zip/take your turn
- Speed limit signs
- Leave a gap
- Slow vehicles/trucks stay in the left lane

Getting the message over could be a combination of:

- Advance preparatory signing – advising the motorist about the type of traffic control that is being approached
- Signing along the merge length
- Signing where needed after the passing lane to restore two lane unrestricted flow
- Reinforcement/education through the electronic and print media, road code, NZTA website

The need for changes to the road layout, in particular road markings also needs to be considered. As a particular request, NZTA asked that European versus NZ merge lane markings be considered (fast lane merge into slow lane, slow lane merge into fast lane).

2.6.5 Design of Potential Layouts

The design of potential layouts is based initially on overseas examples with some adaptation to fit with the existing arrangement of New Zealand passing lanes. The signage considered starts from the existing range of fixed and variable signs in current use, with new signage introduced only where there is no current equivalent or where there is likely to be a significant operational or safety advantage.

The present standard permanent signs for passing lanes comprise: (i) IG 6.1 “PASSING LANE _ KM AHEAD” (ii) G1-6 “PASSING LANE 400M’ (iii) RG-22 “KEEP LEFT UNLESS PASSING” (iv) PW 43-2 road narrows sign with supplementary plate 200m before merge taper (v) PW43 road narrows sign at merge taper.

Provision for variable message signs appears to be specifically allowed currently only for motorways and expressways and temporary traffic control. Extending VMS use to passing lanes may require new or modified land transport rules.

NZTA proposed the following early and late merge layouts for consideration;

- High speed (100 km/h) early merge – (i) advance sign for trucks and towing vehicles to keep left; (ii) “merge here/take turns” signs at 700m and 300m before the desired merge start point

- Medium speed (70-80 km/h) late merge – (i) advance sign for trucks and towing vehicles to keep left; (ii) "stay in both lanes" sign 800 before merge start (ii) "merge here/take turns" sign at 300m before the desired merge start point and separate speed restriction sign; (iii) speed de-restriction sign back to 100 km/h after the end of the passing lane.

Following the results of the simulation modelling, NZTA suggested modifications to provide a more limited number of signs and simplify the messages given to drivers, bearing in mind the conclusion that DEM was more effective than DLM over the whole flow range, and that it would be useful for the VMS sign layout to be able to close the passing lane just before the capacity of the merge is reached.

2.6.6 Cognitive Testing of Potential Layouts

Cognitive testing of the layouts is envisaged as a second stage of the research, given reasonable indications that there are operational benefits in ITS assisted merges. Cognitive testing would use driving simulator facilities such as those of the Traffic and Road Safety Research Group (TARS) at Waikato University. The purpose of the driving simulator testing would be:

- To confirm the general operability of the proposed layouts under a variety of conditions, including curved alignments (vertical and horizontal)[1]
- To test the comprehension of the static and/or VMS signage and the ability of drivers to safely absorb the messages without unsafe interference to the driving task.
- To test variations in the number, type and disposition of signs with the objective of identifying suitable options for field testing

2.6.7 Hardware and Concept Testing

If it is concluded that VMS signs linked to traffic volume and possibly to speed are to be trialled, then testing of the hardware in a non-operational environment will probably be a prudent intermediate stage before deploying in an operational situation. This could first take the form of electronically linking the VMS signs and artificially simulating the electronic outputs from the traffic detectors to test the combined operation of the VMS installation over a peak period. The correct operation of the traffic detectors when linked to the VMS setup would then be checked as a second step.

Ideally, the whole installation would be trialled on a closed test track, prior to deployment in an operational setting. All such research installations are overseas (e.g. TRL). Alternatively the equipment could possibly be set up in New Zealand on a closed or private road, such as a raceway, for initial testing. Creating realistic conditions would require some temporary road marking and marshalling a reasonably large fleet of vehicles and drivers, and either way (overseas or in New Zealand) would be an expensive exercise. A closed trial would allow a range of traffic parameter monitoring to be undertaken that may not be practical under full operational conditions.

2.6.8 Operational Field Trials

Operational field trials are the next logical step, once the operational and economic advantages of ITS-assisted merging have been established through the traffic microsimulation modelling, the road layouts passed as suitable for field trials from the driver simulation testing and the hardware concept proved and tested in either a closed trial, at existing passing lanes with merge problems or

[1] Low light and nighttime conspicuity is also an issue but is not able to be reproduced effectively by the driving simulator.

possibly at work zone locations on four-lane carriageways where the lane in one direction has been temporarily closed.

Field trials will need to be approached with great caution as the road and traffic environments are potentially quite hazardous, involving relatively high speeds with passing and merging manoeuvres. Initial trials should be on relatively straight open alignment on a road section with good safety features (absence of roadside hazards and good recovery distance).

Tests will need to be carried out at relatively high traffic volume and it will need to be decided whether to test early or late merges first. This will probably require a suitable holiday weekend in order to find the necessary high volume conditions. For comparability, field tests should compare three strategies at the same location and for similar traffic conditions: (i) operating the passing lane without any special signing or restriction (ii) closing the passing lane and operating as a single line of traffic (as customarily done on a holiday weekend); (iii) operating with the trial signing.

Advance publicity of the experimental signage will be needed and an independent safety peer review of the layout should be made before the trial. The trial will require NZTA approval and gazetting under the Land Transport Rule 54002 (2004) on Traffic Control Devices Section 3.4, and local advanced advertising.

3 Literature Review

3.1 Introduction

A partial literature review was carried out in developing the project brief, using references provided by NZTA and others located at that stage. Further literature search has resulted in a few more relevant citations. The earlier review has also been included for convenience. We have reviewed the literature under the following general search parameters:

- lane merge behaviour in general
- use of VMS to influence traffic merge behaviour
- perception and responses to VMS, in rural road situations
- 2+1 road layout operation
- use of microsimulation to model passing lane operation and merge behaviour in the presence of VMS

The literature review does not attempt to be exhaustive, but to identify directly relevant experience and theoretical research. This appears to centre on:

- Use of VMS in dynamic lane merging at highway work zones in the USA – in particular the states of Iowa, Michigan, Minnesota and Virginia
- Development of cable barrier divided 2+1 road layouts, originating in Sweden but now used in several North European countries, North America and New Zealand
- "Managed motorways" – use of VMS signing to actively control speed and flow on high volume motorways/freeways.
- Research on driver perceptions and behaviour in relation to VMS in a rural road traffic merge environment.

3.2 Use of VMS for Dynamic Merges at Highway Work Zones

McCoy and Pesti (2001) discuss the development of ITS-assisted merge control strategies at highway work zones in the USA through the 1980 to 2000 period. The drivers for introduction of new strategies for merge control where lane reductions at road work sites require traffic lanes to merge were: (i) to improve merge discipline and the throughput at the merge point; (ii) to reduce driver frustration and "road rage" where people are perceived to be queue jumping; and (iii) to obtain better utilisation of the storage capacity of roadspace upstream of the bottleneck.

The conventional temporary traffic control at the time were "static" forms of merge control, that is fixed signage and markings to warn traffic of the approaching lane reduction, slower traffic, queung and the need to merge left or right.

The review discusses Static Early merge (SEM) strategies citing test simulations **(Nemeth and Rouphail 1982, Mousa et al., 1990)**, the findings of which were that SEM reduced the frequency of forced merging (vehicles push their way into the merged lane), especially at higher traffic volumes, but also increased the average travel times through the zone of effect, because faster vehicles were more likely to be delayed by slower vehicles ahead of them in the open traffic lane. The SEM strategy discourages use of the closing lane for relatively long distances ahead of the lane closure.

The next development was Dynamic Early Merge (DEM), which activates VMS signage instructing vehicles to merge only when traffic volumes rise and/or traffic speed slows to a certain level. The

VMS signage is installed at intervals along the road and which signs are activated depends on the position of the tail of the queue. Field studies by the University of Nebraska **(McCoy et al 1999)** are cited for Indiana.

The Dynamic Late Merge (DLM) strategy encourages drivers to use the closing lane right up to the merge point and then control the merge as a “take your turn” or zip merge. This legitimises the use of the closing lane to offset driver worries about queue jumping and makes better use of roadspace.

Studies by the Pennsylvania Department of Transport **(Orth Rodgers & Associates, 1995)**, showed the system to increase merge capacity by up to 15%. Field studies **(McCoy et al 1999)** also found that there were 75% fewer forced merges and less incidence of “lane straddling” a tactic to prevent following drivers from queue jumping. However, at low traffic flow, the DLM was considered to have potential problems associated with unclear right of way at the merge point and relatively higher traffic speeds.

The paper concluded that DLM was the better strategy but one that should only be activated when traffic flows increased sufficiently to make it a better choice than conventional merging (using the VMS signs to convey warning messages). It was also concluded that for safety reasons speed control should be instituted because of the speed differential between the open and closing lanes.

More recently, **Beacher et (2004)** provided an evaluation of the DLM strategy compared with conventional static control based on the US Manual of Uniform Traffic Control Devices (MUCTD).

The evaluation used both microsimulation using VISSIM and some limited analysis of field situations, although only two sites yielded usable data. 2 to 1, 3 to 2 and 3 to 1 merges were studied, but only the first is pertinent to this study. Various factors were considered in the simulation analysis including the percentage of trucks, hourly volumes, side of lane closure and speed limit.

The throughput or capacity of the merge was the main evaluation criterion. For a 2 to 1 merge, the simulation found a very small difference (0.4%) in throughput capacity for DLM compared with SEM.

The DLM strategy gave higher throughput for a higher percentage of heavy vehicles (i.e. was able to pass slow vehicles more effectively upstream of the merge). The limited field tests were in general agreement with the simulations.

A more recent simulation study introducing a further development of the ITS merge-control concept is reported by **Yang et al (2009)**. The concept is lane-based signal merge (LBSM) to create a queuing length upstream of the merge, a weaving area and then the merge point. The signal controls would be activated once traffic volumes reach a critical level. The concept works in a similar way to ramp signalling, metering traffic into the merge area at a rate that allows the merges to operate efficiently and at least delay.

The study was entirely simulation based, and compared LBSM against conventional merge (SEM), DEM and DLM, using VISSIM as the simulation platform. Sensitivity to traffic signal cycle length and approach volume were tested.

The results showed all merge control types to be superior to DLM while operating below the capacity of the merge. Once the capacity point was reached, LBSM showed the greatest throughput at about 1,800 veh/h, followed by DLM at about 1,600 Veh/h, then conventional merge and finally DEM at around 1,400 veh/h. Comparing total delay through the system, the ranking was the same, with LBSM producing the least delay per vehicle once the merge capacity was reached.. At volumes below merge capacity, DLM had the advantage, but a lower upper limit on capacity compared with LBSM. The minimum average delay of the LBSM layout was obtained at a signal cycle length of 2 minutes.

Another recent study of DLM **(Sperry et al, 2009)** was for the Iowa Department of Transportation. Unfortunately, due to equipment malfunction and the limited periods at which traffic volumes were high enough to activate the VMS, relatively little data was obtained, and therefore was not able to show if there was any statistically significant changes in merging behaviour or overall benefit from the DLM system.

3.3 2+1 Cable Barrier Road Layouts

Roads with continuous alternating passing lanes are a logical intermediate capacity stage between periodic passing lanes and four lane roads. Originally, three lane roads with simple lane lines providing overtaking opportunities in either direction were used extensively in the UK. These were later thought to be unsafe and various systems of lane marking in single or double, continuous or broken white or yellow lines, were introduced to separate the opposing streams and provide for safe overtaking. Three lane roads with alternating two and one lane sections, referred to as 2+1 layouts have been widely introduced in Sweden and elsewhere in northern Europe; in Sweden there are now over 1,800 km in operation **(Carlsson 2009)**. The modern European systems operate with or without median barriers and with various forms of transition at the end of each passing section. In recent years 2+1 roads have been introduced in Ireland and New Zealand. We have not located any examples where ITS control has been applied to 2+1 layouts to enhance capacity.

The **Transportation Research Board (2003**) reviewed practice in Finland, Sweden and Germany for potential application in the US. At that time, 2+1 roads had only recently been introduced. The 2+1 system was recommended for introduction in the US on roads where passing lanes provide insufficient passing opportunities but where volumes do not warrant four lanes. Again, the safety benefits were a primary driver for introduction. Their use was proposed for level to rolling terrain rather than for mountainous terrain or where prolonged gradients make crawler lanes a more appropriate option. The lengths of the two lane sections were proposed to be 500m at a directional flow of 100 veh/hr to between 1,000 and 2,000m at 700 veh/hr.

While some exceptional examples of 2+1 roads operating at 1,900 veh/h were observed, generally the capacity was much lower, around 1,600 to 1,700 veh/h in Sweden and in Finland congestion at the merge became apparent at directional flow rates of 1,200 to 1,400 veh/h. For US applications, flow rates up to a maximum of 1,200 veh/h were proposed, compared with 1,700 veh/h for two lane highway capacity based on the Highway Capacity Manual.

Carlsson (2009) summarised Swedish experience with 2+1 lane roads. Their safety record has been found comparable to motorways in terms of fatality rate, and a 75% improvement on conventional two-lane rural roads.

The directional capacity of 2+1 roads was found to be determined by the capacity of the merge, which was 1,600 to 1,700 veh/h over a ¼ hour period or 1,500 to 1,550 veh/h over the peak hour. The breakdown of stable flow was found to occur rapidly near the capacity point. This capacity was 300 veh/h below the capacity of a conventional Swedish two-lane expressway (13m traffic lanes plus 3m paved shoulders on a high standard alignment with interchanges and side access control). The main incentive for introducing three-lane layouts was therefore to improve safety within the available road formation width without a significant sacrifice of capacity.

Traffic speeds were found to improve by 2 km/h on 90 km/h posted sections and to remain the same on 110 km/h posted sections. For driver travelling on highway sections with a 90 km/hr speed limit, the 2+1 layout enabled vehicles to travel slightly faster. For drivers on a state highway with a 110 km/h speed limit, drivers were already travelling close to the operational limit of the road section, which was already slightly above the speed limit. Therefore, there was no marked change for 2+1 lane road sections with 110 km/hr speed limit.

3.4 Managed Motorways – Use of VMS for Active Speed and Lane Control

In the UK, the concept of "managed highways" is gathering momentum. This refers to VMS control of traffic lanes to actively manage lane operation during peak periods and in response to traffic incidents. At present, the main applications appear to be on full motorways **(Department for Transport, 2008)**, with the additional capacity provided by peak period use of the hard shoulder as an auxiliary lane. Segregation of traffic streams, such as high occupancy vehicle (HOV) lane control or reserved bus and truck lanes are another application.

While there have been simulation studies for conversion of two lane roads to 2+1 operation in the UK, none of these appear to have involved any form of ITS management.

3.5 Driver Perceptions and Behaviour

Harder and Bloomfield (2008) researched the effectiveness of changeable message signs using a driving simulator. Some drivers were found to reduce their speed in the presence of message signs.

The general findings were that the clarity and brevity of the message influenced driver slowing behaviour and responsiveness to the message. The message content was not related directly to lane closure information but to other traffic and more general incident messaging.

Nygårdhs and Helmers (2007) studied the interaction between driver behaviour and VMS. Some key findings were:

- VMS to sign a speed reduction – must be accompanied by a credible reason to be effective.
- Forewarning of an important message ahead is an effective way of getting driver attention; more than one such warning is desirable.
- Real time messages must be accurate and up to date or else VMS loses credibility
- Fixed rather than flashing or running VMS signs are best, and should be light emitting technology rather than reflective.
- A data bank of short messages covering all likely situations should be used; long or nonstandard messages are harder to absorb.
- All VMS messages should be direct and relevant, recognising that their impact and conscious perception will reduce the more ubiquitous they become.

Marketline Research (2003) carried out market research for the Minnesota Department of Transport on driver perceptions and attitudes towards signage in work zones. The research was based upon driver visualisation of a recent journey. Some of the key findings were:

- When faced with an advanced sign warning of a lane closure and request to merge, more drivers would merge right away in light traffic than in heavy traffic (41% compared with 34%).
- In heavy traffic, the majority of drivers would be opportunistic in merging, continuing in the closing lane looking for an opportunity to merge rather than merging immediately; only 3% would wait until nearly at the lane closure.
- The mention of slow traffic ahead was a stimulus to merge early, and portable signs encouraging early merge would give higher compliance.
- 75% of drivers would make space for merging vehicles; 23% would block merging drivers as queue jumpers and one third of these would express feelings of anger; about 15%

admitted to straddling lanes to block other drivers from making later merges than themselves.

- 93% thought that real time messaging have value, but some indication that the installation is live and responsive is needed.

3.6 Summary of Literature Review

Summarising the literature review, our main findings are:

- Capacity - Studies of the capacity of 2 to 1 lane merges indicates that both ITS assisted and unassisted layouts can be expected to have a lower throughput than a single traffic lane.
- The capacity point is at a sustained directional flow rate in the range of 1,400 to 1,600 veh/h compared with a single lane capacity of around 1,700 veh/h/lane, although this will be influenced by the percentage of heavy/slow vehicles and by free flow traffic speeds. Near the capacity point, the breakdown of stable flow is likely to be quite sudden and cause a tailback until the demand flow drops sufficiently to clear the queue. This capacity drop of 100 to 300 veh/h/direction between single lane flow and a 2 to 1 merge is the basis for the practice of closing off passing lanes in holiday traffic peaks.
- Benefits of merge strategies – the time saving benefits of early and late merge strategies lie in the extent to which they can lead to a more orderly merge than an unassisted strategy at flow rates below capacity and increase the flow that can be accommodated. The range of capacity increase between conventional and dynamic late merge may be as high as 200 veh/h where there is a high percentage of trucks/slow vehicles, but previous research has not been conclusive on this point. A novel idea is lane-based signal merging when the single lane free flow capacity is reached, similar in concept to ramp signalling.
- ITS Applications - VMS merge assistance has been used for temporary traffic control through road work zones and for “managed motorways”. However, we did not locate any comparable application to an ITS assisted passing lane of 2+1 road layout.

4 ITS Layouts

4.1 Introduction

The forms of ITS most likely to be applicable are variable traffic signs (VTS) linked to directional flow volume and/or speed using buried loop detectors. VTS can include (i) Active Warning Signs incorporating LED symbols, messages and optionally flashing corner lights; (ii) Variable message Signs (VMS) displaying character messages in up to three lines, preferably full matrix which can be used alone or in conjunction with other signs; and (iii) Changeable Message Signs (CMS), fixed signs incorporating variable electronic elements.

The assumption is that the test configurations will be permanent installations and not portable signs for occasional roadside deployment.

As the lanes will be operating at relatively high flow rates, it may not be sufficient to rely on roadside signage to control overtaking traffic as this may not be easily visible to traffic in the passing lane. The Rule on TCD 4.4(9) requires signs to be mounted on the left or above the traffic lane apart from exceptional cases. So options of both roadside and overhead-mounted signs should be considered.

4.2 Available Existing Conforming Signs

The available existing conforming signs which may be used comprise:

- Roadside left-mounted PW43 road narrows ahead permanent symbol sign (right hand narrowing), and with advance PW43.2 with “200m” (distance normally signed) – there is no current provision for a variable version of the PW43 sign.
- Overhead variable lane control signs – red cross, green arrow or yellow diagonal arrow over the passing lane indicating the need to move to the left hand lane; Land Transport Rule 7.13 on Traffic Control Devices requires this form of variable signing to be overhead of the lane.
- If a reduction in the speed limit is required, variable R1-2.1 regulatory sign, highest speed limit, red lit roundel and changeable speed value in yellow or white with optional corner flashing lights.
- In advance of the passing lane a R7-10.1 general regulatory VMS saying “TRUCKS AND TOWING VEHICLES USE LEFT LANE” or similar (will need consideration of shortest message to convey the meaning).
- Other conforming advisory or regulatory VMS deployed on the roadside or above the traffic lane, on their own or in conjunction with other signage; such as “USE BOTH LANES”, “MERGE LEFT, NO PASSING”, “MERGE HERE, TAKE YOUR TURN”, etc.
- Again, if there has been a reduction in the speed limit, a R1-3 fixed derestriction sign following a variable speed limited passing lane – there is no current provision for the derestriction sign to be a variable sign.
- Where a passing lane is closed at rates of flow above the merge capacity, a temporary lane closure sign similar to the message shown on truck-mounted VMS signs with left to right moving arrow. This sign has the advantage of already being widely recognised.

4.3 Signs Used in Overseas Examples

Static early and late merge examples are not relevant to permanent road layouts such as passing lanes and 2+1 lanes. The static merge relies on additional advance warning signs of lane closure which is only relevant to a temporary layout such as a work zone. Static warning signs already exist at passing lanes.

The dynamic early merge example is the Indiana Lane Merge System (described in Beacher *et al* 2004), "DO NOT PASS WHEN FLASHING" fixed signs with flashing side lights mounted at intervals on the same side of the road as the lane restriction (the illustration is for median mounted signs and left (overtaking) lane closure); the signs are linked to queue detectors – as there is no median in the passing lane layout, this static merge signing would be less effective and possibly inappropriate if mounted at the roadside when there is a heavy traffic stream that shield the view to traffic in the passing lane.

The dynamic late merge concept piloted by Pennsylvania DOT (McCoy and Pesti, undated) employed an advance "USE BOTH LANES TO MERGE POINT" sign and "MERGE HERE TAKE YOUR TURN" sign at the merge point on each side of the carriageway near the merge taper. While tested as a static system, the dynamic concept is that the signs could be VMS and traffic responsive, activated only at high flow. Minnesota DOT has issued guidelines for a dynamic late merge system for work zones. Similar VMS is used plus a "STOPPED TRAFFIC AHEAD – USE BOTH LANES" sign before the end of the queue.

In both early and late merge concepts, the lane closure itself is also signed with move left/right arrows at the taper point.

Considering the overseas examples studied to date and NZTA's suggestions in the brief, the following dynamic early and late merge configurations were considered. The early merge concept was assumed to operate at 100 km/h speed limit and the upper flow limit was expected to be lower than for the late merge case.

4.3.1 Dynamic Early Merge – Configurations for Testing

(i) At the 400m advance warning sign of the passing lane add a traffic actuated VMS "TRUCKS AND TOWING VEHICLES USE LEFT LANE" or similar;

(ii) At the start of the passing lane replace the fixed sign "KEEP LEFT UNLESS PASSING" with a VMS showing the same message or "TRUCKS AND TOWING VEHICLES USE LEFT LANE" or similar;

(iii) Upstream of the merge taper start a left-mounted VMS "OVERTAKING TRAFFIC MERGE LEFT" or similar; alternatively an overhead-mounted left to right diagonal arrow; the distance from the merge taper start to be tested but probably between 800m and 300m;

(iv) Upstream of the merge taper start a left-mounted VMS "MERGE HERE, TAKE TURNS" or similar; alternatively an overhead-mounted left to right diagonal arrow; the distance from the merge taper start to be tested but probably between 300m and 0m, the distances linked to (ii) and with the passing lane length.

4.3.2 Dynamic Late Merge – Configurations for Testing

(i) Similar advance warning signage as for the early merge, or "TRAFFIC CONGESTION, USE BOTH LANES" at the passing lane start;

(ii) Upstream of the merge taper start a left-mounted VMS “TRAFFIC CONGESTION, USE BOTH LANES” or similar; the distance from the merge taper start to be tested but probably between 800m and 300m;

(iii) Consider the need to supplement this with a variable regulatory speed sign (R1-2 or R1-2.1) based on the traffic volume will be considered. In this case the speed sign may alternate with the VMS message.

(iv) Upstream of the merge taper start a left-mounted VMS “MERGE HERE, TAKE YOUR TURN” or similar possibly with supplementary left to right arrow or the VMS alternating with the arrow; the distance from the merge taper start to be tested but probably between 300m and 0m.

(v) Variable regulatory speed sign (R1-2 or R1-2.1) downstream of merge taper end to sign 100 km/h when the dynamic late merge is in operation.

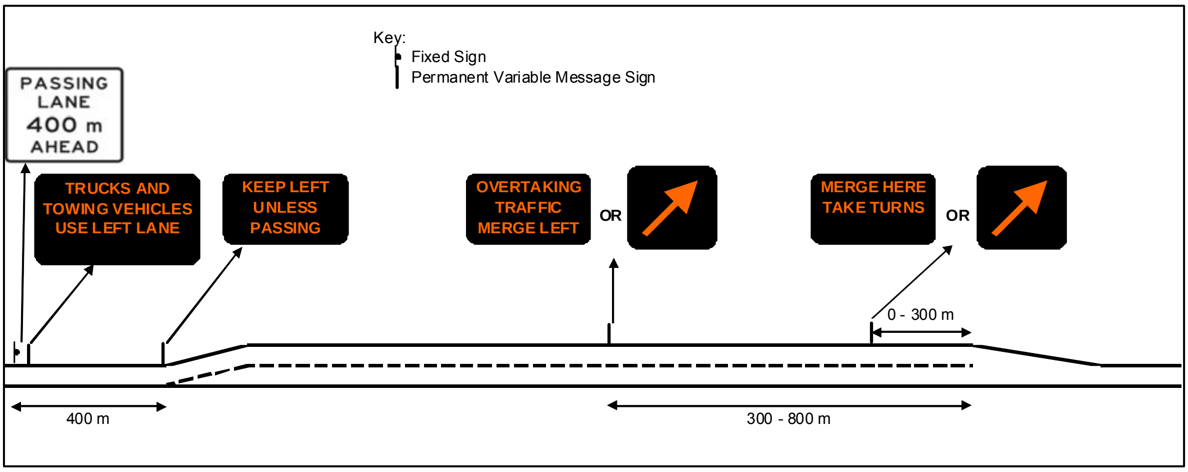


Figure 3 – Dynamic Early Merge, Diagrammatic Layout

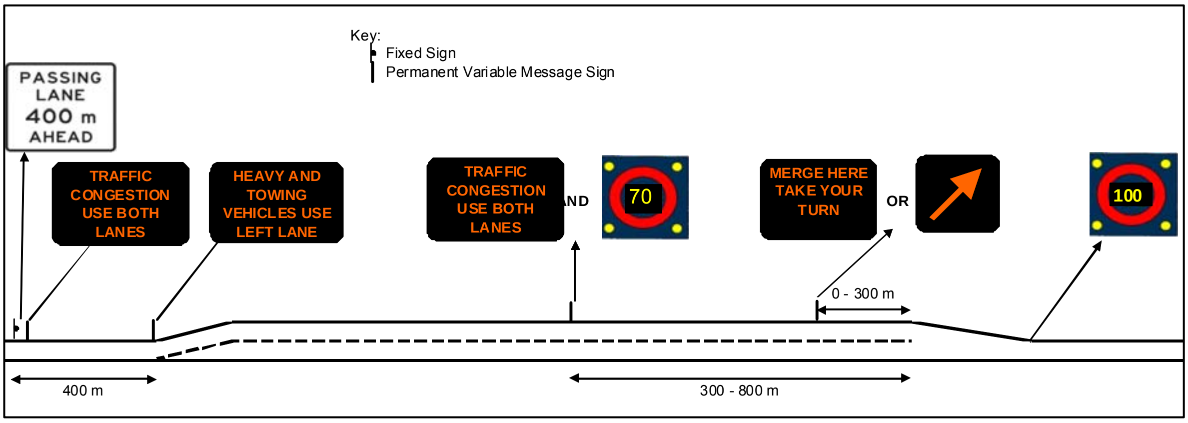


Figure 4 – Dynamic Late Merge, Diagrammatic Layout

4.4 Signage for Early or Late Merge Based on Traffic Flow

For maximum flexibility and use of a VMS layout it would be advantageous to be able to use an early merge strategy at moderate volumes possibly changing to a late merge strategy at high volumes, such as over-capacity situations in seasonal peaks. In this case, an optimum positioning of the VMS signs to cater for both cases would be needed comprising (i) advance signage ahead of the passing lane (ii) signage at the passing lane diverge start (ii) signage at two intermediate points upstream of the merge taper (iii) signage at the merge taper start (iv) 100 km/h speed sign downstream of the passing lane.

4.5 Lane Markings

Driving simulation research has demonstrated how lane markings at the diverge and merge tapers can influence lane use (**Baas and Charlton, 2005**). The present NZ treatment of a continuity line at the diverge taper that directs the flow into the left lane was shown to result in more vehicles using the left lane rather than continuing straight on in the overtaking lane, when compared with the pre-existing treatment without the continuity line. The July 2000 changes also introduced diagonal shoulder stripes at the merge taper.

NZTA requested that the influence of European versus NZ markings be included in the modelling of with- and without-ITS assistance merges. An essential difference between NZ and European treatment is that in NZ the slow traffic is required to move left into the auxiliary lane and then merge back into the overtaking lane at the end of 2 lane section, whereas the European layouts require the overtaking traffic to move across into the auxiliary overtaking lane and then merge back into the slow lane, creating a higher speed merge.

There are also other differences. The Swedish 2+1 lane markings are similar to the pre-July 2000 New Zealand treatment, starting at the end of diverge taper and stopping just after the start of the merge taper when the overtaking lane has narrowed to 2.5m, but also add merge arrows in the overtaking lane ahead of a merge taper. The German 2+1 treatment, without cable barrier, continues the overtaking lane full width up to the end point, where the overtaking lane ends in a hatched zone providing a separation from the opposing direction. Merge arrows are used similarly to Sweden. Finland uses a merge taper but carries the lane line through to the end of the taper, as Australia, and includes merge arrows.

Carrying the lane line through to the end of the taper was found by **Baas and Charlton (2005)** to slow traffic in the overtaking lane as vehicles tried to find gaps to remerge. It would appear that stopping the lane line at the start of the merge taper plus merge arrows would provide the best marking incentive for an orderly merge; also that the current diverge continuity line treatment would reduce the number of slow vehicles using the overtaking lane.

So the comparison between NZ and European markings involves both the intrinsic difference in layout of the slow versus fast lane merge together with the differences in continuity lines, arrow markings.

At the diverge point, this becomes a question of how the modelling can be adjusted to reflect a higher or lower use of the left lane by slower vehicles in response to the combined influences of ITS signage, requirement to shift left (or not), and with or without the continuity marking.

At the merge point it becomes a question of how the modelling can be adjusted to reflect the high or low speed merging lane and how the merging is affected by addition of painted merge right arrows. (We suggest that continuing the lane line through to the end of the taper not be considered).

4.6 Subsequent Refinements to the Layout

Subsequent to the results from the simulation modelling, NZTA has indicated that it will be considering a more simplified layout than the two above, possibly involving a VMS sign with merge message about 500 m upstream from the merge (one each side) and a VMS sign with alternating “passing lane closed”/”straddle both lanes” messages by the diverge (one each side). This is discussed further later in the report.

5 Simulation Modelling of ITS Merge Strategies

5.1 Comparison of Micro-simulation Software

There are three widely used micro-simulation software tools AIMSUN, PARAMICS and VISSIM. These have been developed mainly for urban roads and networks, and for multi-lane highways, not specifically for rural two-lane roads and for passing opportunity analysis. The literature review has shown up some limited use of micro-simulation for the two lane rural highway context. Both VISSIM **(Ning Yang et al, 2009**) and PARAMICS (**Lanyu Chu et al, 2005)** have been used, the former to simulate the operation of lane signalling in an over-capacity work zone situation and the latter mainly for a route choice model where there is an alternative route. However, neither are directly comparable to this research.

Kourey (2002) made a high level survey of micro-simulation software developed specifically for passing lanes, noting that TRARR which has the best history of use in, and calibration for, New Zealand conditions, also reviewing the American TWOPASS software which is similar in concept but little used in New Zealand. While TRARR and similar models (there are one or two others) are well suited to passing lanes and to traffic stream interaction with rural road geometry operating at relatively uncongested flow levels, they are not suited to congested traffic, bottlenecks and where there is some form of directive signing.

So the choice of software for this study lay between AIMSUN, PARAMICS and VISSIM. We compared the suitability of each package in Table 1

Table 1 – Comparison of Microsimulation Modelling Software for ITS-Assisted Merging

General Attribute	Specific Attribute	AIMSUN	VISSIM	Paramics
Geometrics	Urban environment	2	2	2
	Rural environment	1	1	1
	Weaving / Lane-changing / Overtaking	2	2	2
	Gradient	1	2	2
	Early merge at Passing Lanes	2	2	2
	Late merge at Passing Lanes	1	2	2
	Direct modelling of lane markings	1^{+}	0	0
	Indirect modelling of lane markings (NZ vs European) through controlling priority at the merge	1	2	1
Transport Modes	Vehicle types	2	2	2
	Pedestrians and bicycles	1	2	1
	Buses	2	2	2
	Light Rail	1	1	1
	Bus / Cycle / HOV / Heavy Vehicle priority lanes	2	2	2
	Slow vehicle / Overtaking lane	2	2	1
Traffic Management	Variable message signs	2	1	1
	Lane-closure	2	2	2
	Incident modelling	2	1	2
	Capacity reduction (indirectly available)	2	2	1
Assignment	Static	2	2	0
	Dynamic	2	2	2
Input / Output Features	Input data requirement	2	2	2
	Output file formats (accessibility)	2	1	1
	Diversity of data output: data by individual vehicle	1	2	2
	2-D / 3-D Animation	1	2	1

General Attribute	Specific Attribute	AIMSUN	VISSIM	Paramics
Ease of Use	Input effort (data requirement e.g. 2 = minimal, 1 = more effort required)	2	1	1
	Software envionment	2	2	1
	Completion effort	2	1	1

Key: 2 = Good; 1 = Average; 0 = Not Available

Note: + limited to modelling of no lane-changing through the implementation of solid white lines

There was no factor that completely ruled out any of the three software programs. However, they were all found to have some limitation in different areas. AIMSUN had the advantages of ease of use, allowing different scenarios to be set up and modified more easily than the other two packages. It also had a more accessible output file format, so that post-processing of simulation output data by another programme such as Excel would be easier to achieve. AIMSUN also provided for solid white lines to be used as a control on lane behaviour, a feature that the other two packages did not have. A useful feature was the direct modelling of responses to VMS available in AIMSUN; while the behaviour could be reproduced in the other packages it required more programming.

AIMSUN's animation features were not as good as VISSIM, but this was not really important for this study. A slight drawback was that obtaining output data on an individual vehicle basis, which may be required for some data analysis, was not as easy as in VISSIM. Also AIMSUN did not handle gradients as well as the other two packages – which was not an immediate problem as the test simulation assumed flat grade, but limited the usefulness of AIMSUN to test gradient effects. More importantly, modelling of late merges was not as flexible as in VISSIM.

VISSIM had the advantages of more flexible and diverse data output, but somewhat lower ease of use than AIMSUN. A positive feature was the ability to replicate zip merges and generally to exert greater control over merge behaviour. Of all the packages VISSIM was considered the most adaptable to the lane control and merging requirements of this study.

PARAMICS was much less flexible in data inputs and outputs and less easy to use in set-up time for a simulation, all of which counted against it for use in this research study. As it did not have any features that made it superior to VISSIM, we ranked the suitability of the software as AIMSUN and VISSIM equal first but with different advantages/disadvantages and PARAMICS third.

5.2 Initial Test Modelling

Test models were set-up in both AIMSUN and VISSIM to understand the sensitivity of model parameters to vehicle passing and merging behaviour in a rural environment, and the feasibility and level of accuracy of modelling early merge, late merge and NZ versus European lane markings at the merge point (slow lane merges into fast lane or vice versa).

A comparison of the software showed the following points in common:

- Both packages were able to demonstrate a breakdown at the merge under unassisted conditions, which causes a tailback effect at high flows.
- The vehicle behaviour was highly dependent on the fleet size distribution, in particular the vehicle length, acceleration and deceleration rates. There was limited current NZ data to support the comprehensive distribution of parameters required by the software, and in such circumstances, the default parameters were the best available source.
- Different vehicle classes with different driver behaviour (aggressiveness) could be specified

- Although both packages had different built-in algorithms to determine how vehicles behave at passing lanes, the on-screen simulation performance was observed to be fairly similar and there was no indication that the models produced unrealistic behaviour.
- An 'early merge' situation could be realistically represented in both packages through varying the 'distance to a merge' parameter.
- Neither package could model the effect of lane marking directly, but by setting the driving rule to ' keep left' and having the auxiliary lane on the left (or right), we were able to replicate a slow speed merge or a high speed merge situation

Points of difference between AIMSUN and VISSIM were found to be:

- For a 'late merge' situation, there was somewhat more ability to model the merge behaviour in VISSIM than in AIMSUN, although neither package could directly control merging as far as directing an alternate vehicle zip merge.
- Under ITS assisted conditions, AIMSUN had the advantage of incorporating VMS signs directly into the program settings, where vehicles' compliance to this guidance indication could be varied whereas, in VISSIM, the VMS had to be modelled as a 'compulsory' message coded to a section of the road.
- In very high flow unassisted situations, vehicles could become stuck at the end of a passing lane in VISSIM because finding a gap was difficult and vehicles could be lost from the simulation; whereas, AIMSUN handled this situation by forcing vehicles to merge.
- AIMSUN was poor at handling gradients, and this could pose a difficulty in any extension of the simulation to consider gradient effects.
- VISSIM had been used more frequently for this form of research in the international literature, and was generally a more widely used software programme than AIMSUN.

While the choice of software was finely balanced, VISSIM was selected because of its more widespread use internationally, somewhat easier extraction of raw data, ability to handle gradients if needed, and slightly better control over a forced merge.

5.2.1 VISSIM Base Case Specification

Because of the large number of potential variables, the approach taken was to specify a representative “base case” to test for different signage and marking configurations and then test the sensitivity of varying each of the main parameters in turn.

The “base case” was selected as a 1250m passing lane. Due to the limitations of the simulation modelling, the single lane sections in advance and downstream of the passing lanes had to be specified as no overtaking, so questions of directional split of traffic volume did not come into consideration.

Each test simulation was run over a range of traffic volumes and with 10% heavy commercial vehicles (HCVs). The directional traffic volume steps were 250, 100 veh/h and in some cases 50 veh/h as the layouts neared capacity. Traffic volumes from 250 up to 1750 veh/h were tested.

In the simulation modelling, the effects of the VMS signage were modelled as:

- Use of the left lane by slow vehicles (dynamic early merge) – was modelled by splitting the speed distribution and assigning the lower half to the left lane and the upper half to the right (passing) lane; the sensitivity testing varied the proportions of the speed distribution in each lane;

- Use of both lanes by all traffic (dynamic late merge) – was modelled as 50% assignments of arriving vehicles to each lane, with zero lane changes permitted; the sensitivity cases provided (a) 60%/40% initial assignment to roadside and centre lane and (b) allowed weaving following this initial assignment;
- Lane changing in VISSIM was controlled by a defined distance ahead of a fixed point at which vehicles begin to attempt to change lanes; the fixed point was the start of the merge taper and the distance ahead was the position of the VMS merge sign;
- Speed sign observance – each link could have a legal speed, so a VMS speed sign could be modelled by changing the link speed accordingly. The observance of the speed sign was controlled by the distribution of desired speeds around the posted speed.

To avoid the complications that may be introduced, the research focused on passing lanes that did not include significant horizontal curves or crests. If the concepts were proven to be feasible on relatively straight level road sections, then extension to more winding alignment could be addressed at a later date.

The base case and the road layouts considered are summarised in Table 2 below:

Table 2 – Microsimulation Test Configurations

PL Configurations
1. no PL or PL closed
2. unassisted PL
3. dynamic early merge (DEM)
4. dynamic late merge (DLM)
3a. DEM + Euro markings
4a. DLM + Euro markings

5.2.2 Platooning Ahead of the Passing Section

The simulation was set up as a single lane upstream starting length, the two lane passing section, and a single lane downstream length. There was no passing other than on the passing lane section. The vehicle simulation was fed in at the start of the upstream length, and was programmed as a random Poisson arrival distribution (exponential spacing). The extent of platoon formation immediately upstream of the passing lane therefore depended on the length of this upstream section.

The base case used a 5,000 m upstream length, which generated a platooning pattern and range of percentage following vehicles at the start of the passing lane, depending on the hourly traffic volume. The sensitivity of the simulation results was tested to reductions in the percentage following by varying the upstream length.

Downstream of the passing lane, there needed to be sufficient length for effects of the passing lane to dissipate, so that with- and without-passing lane options could be compared in regard to road user costs. At the relatively high flow rates being tested and with no downstream passing opportunities a 5,000 m length was expected to cover all cases.

5.2.3 Peaking Characteristic and Tail Back Effects

The micro-simulation could assume either a uniform flow rate over the simulation period or a peaking profile. Tail-back (over-saturation) effects occurred once the demand flow exceeded the capacity of the merge. Once flow instability occurred the capacity dropped back to the forced flow rate, and a rapid development of a queue occurred. The tail back then increased until the demand flow reduced to a level that allowed the forced flow capacity at the merge point to dissipate the queue. As the forced flow rate was normally lower than the practical capacity of the merge, once a breakdown in flow did occur, it would require the demand flow to drop well below the practical capacity before the queue started to dissipate, which could be a long time.

If a uniform flow rate were modelled, then the micro-simulation would be able to identify the flow rate at which the demand flow exceeded the capacity of the merge. For any particular layout, successive simulations at higher hourly flow rates would show a gradual decrease in mean vehicle speed following the PL until the capacity point was reached, at which point the mean vehicle speed would drop to the forced flow rate.

If a peaking characteristic were modelled, then the onset and dissipation of the queue, and the length of tail back, would be determined by the peakiness of the traffic flow. For the purposes of testing and comparing options, we used uniform flow and ran the simulation for an initial 15 minute warm up period followed by one hour of simulation.

Whether or not a tail back occurred then depended on whether the demand volume exceeded the capacity of the PL merge at any particular flow rate. If so, then a queue formed and extended over the duration of the simulation. The rate of increase in the queue length could be calculated from the difference between the demand and capacity flows, and the distance headway between queued vehicles. The queue formation could also be observed during the simulation. Comparison between the observation and calculation were found to be in good agreement.

5.2.4 VISSIM Model Adjustment

A number of the default parameters in the VISSIM model required adjustment to replicate the vehicle speed distribution, vehicle following distribution and lane changing behaviour observed in field studies. Sources used in the model adjustment to field conditions were **Cenek and Lester (2008)**, **Koorey and Gu (2001)**, and **Bennett (1985)**. Details of the parameter adjustments are in Appendix A.

Two modelling parameters that were particularly sensitive in respect of the merge behaviour were the desired time headway between following vehicles (CC1 in seconds) and the variation in this parameter (CC2 in metres). The default values for these parameters (0.9 for CC1 and 4m for CC2) did not reproduce the distribution of time headways over the range of headways at which vehicles were regarded as being in platoons, from a little below 1 second up to 4 seconds. The calibrated model parameters were 1.45 seconds and 10 metres.

Another parameter that required some significant adjustment was the speed distribution for light and heavy vehicles. More details on model adjustment are given in Appendix A.

5.3 Base Case Modelling Results

In the following comparison, the DEM and DLM layouts assumed NZ markings at the merge, that is the slow lane moving right to merge into the fast (overtaking lane), rather than the European markings, which are discussed later. All simulations were replicated at least five times and the results averaged. At flow rates approaching capacity, the variation between simulation runs was greater, and the replication was increased to 10 times.

5.3.1 Platooning Upstream of Passing Lanes

As noted above, the degree of platooning and percentage of following vehicles is influenced by the run-in length of the simulation model, initially set at 5,000 m, and the hourly one-way volume. Table 3 shows the percentage of following vehicles (based on 4 second headway) at various demand flow rates and comparative figures from the **Cenek and Lester (2008)** research. There are no field results for the higher rates of flow. Two of the field sites 2e and 4j are in flat terrain and with a percentage of heavy vehicles in the 10 to 13% range. Site 6e is in mountainous terrain, so is not as comparable, but is the only site with percentage following results for a higher flow rate. We have not examined the passing sight distance availability upstream of the two sites for comparability of the 5,000m run-in length in the simulation with the conditions in the field research.

It can be seen that, based on a 5000m length of no passing ahead of the passing lane, the flow is heavily platooned when it reaches the passing lane at flow rates at which the ITS merge strategies were being tested.. At the 250 veh/h one-way flow rate, the simulation and field research results for percentage following were similar. For site 6e, the field results at 350 veh/h and 700 veh/h one-way were both lower than the simulation for percentage following but the site being in mountainous terrain was also less comparable (including use of a 4 second headway criterion for following vehicles). It should be noted that the simulation model was not calibrated to replicate the percentage following in the field research; the comparison is shown to demonstrate reasonable alignment of the simulation with the limited available field data.

Table 3 – Base Simulation, Percentage Following Vehicles at Start of Passing Lane

Demand Flow veh/h	% Following Simulation	Field Research (Cenek & Lester 2008)		
	5,000m	Site 2e	Site 4j	Site 6e
150	-	32.3%		
250	46.8%	45.0%	45.8%	
350	-		51.1%	45.0%
500	70.0%			
700	-			~67.0%
750	81.3%			
1000	87.9%			
1250	92.5%			
1400	94.2%			
1500	94.8%			
1600	95.4%			
1700	95.5%			

5.3.2 Operational Capacity

The flow rate upstream of the passing lane and the output flow rate immediately following the passing lane were used to identify the merge capacity and how merge capacity diverged from the demand flow as the capacity point was approached. The plots show the measured (model input or demand) flow on the y-axis and the flow rate just before and just after the passing lane on the x-axis.

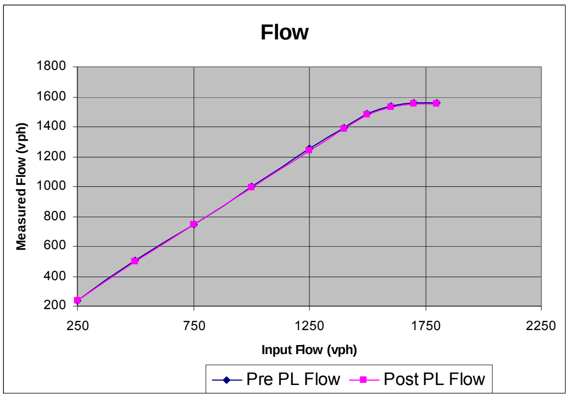


Figure 1a – Closed PL, One-Way Flow

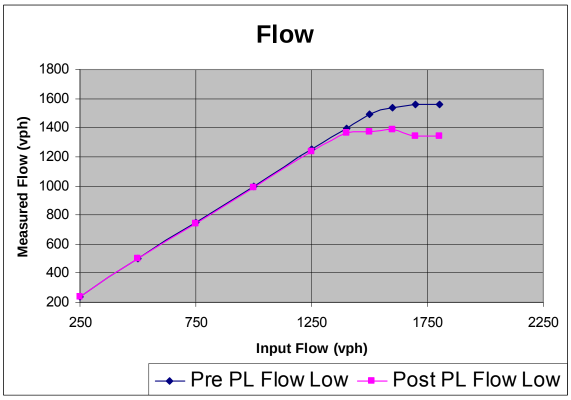


Figure 1b – Unassisted PL, One-Way Flow

The first plot 1a shows the passing lane closed, that is single lane flow. The demand flow and throughput capacity plots completely overlap in this case. The demand and output flows are equal up to 1500 veh/h one-way, at which point the flow on the single lane section starts to tail off to a maximum throughput capacity at just under 1600 veh/h one-way. In plot 1b, unassisted passing lane, the flow upstream of the passing lane behaves as before. Immediately downstream of the passing lane the flow shows that the capacity of the merge breaks down at a demand flow of 1500 veh/h one-way, following which the throughput capacity drops to 1400 veh/h one-way.

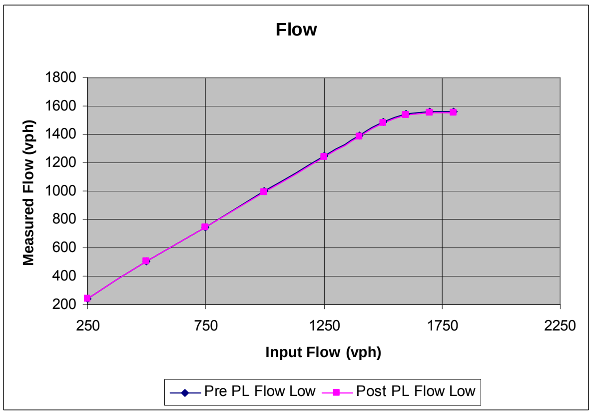


Figure 1c – ITS Early merge, One-Way Flow

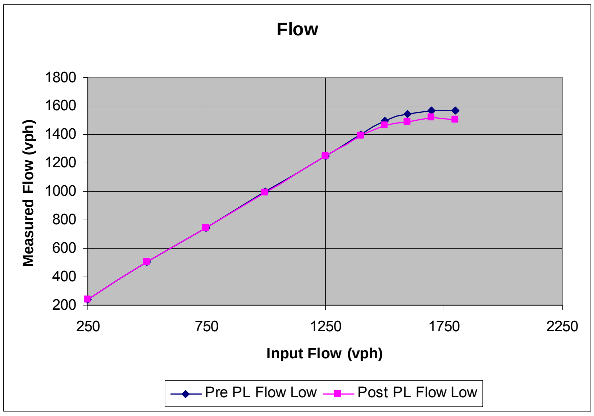


Figure 1d – ITS Late Merge, One-Way Flow

In Plot 1c for ITS-assisted early merge, the simulation results indicated that the early merging strategy had brought the throughput capacity of the merge back up to that of the single lane option 1a, just below 1600 veh/h one-way. This result was sensitive to the vehicle following calibration of the model. Using the default calibration, which involved platooned vehicles following more closely than observed in field research and more appropriate to a multi-lane highway/motorway situation, the throughput capacity of the merge was less than the single lane flow in 1a but better than the unassisted passing lane 1b.

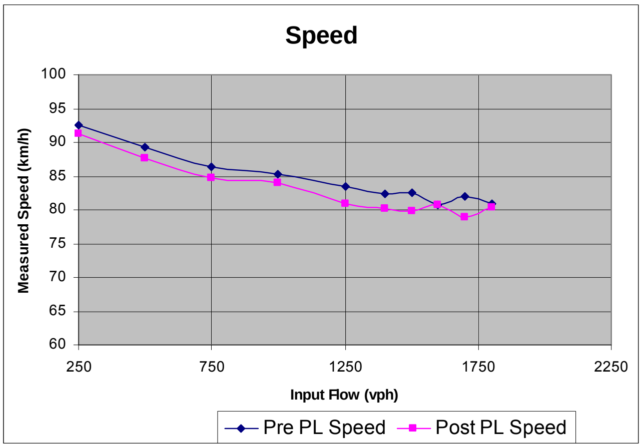
In the late merge Plot 1d, the simulation showed the throughput capacity to be about 1500 veh/h one-way, lower than the early merge but better than the unassisted passing lane. In both 1c and 1d the base case assumes no speed restriction, i.e. the open road limit of 100 km/h prevails. There was no evidence from the modelling that the early merge breaks down at demand flows below that in the late merge. In fact the DEM appeared equal or better than both the unassisted and DLM cases across the flow range and equal to the closed passing lane option. The output flow rates compared against demand flow were as follows.

Table 4 - Base Case, Modelled Flows Downstream of PL, veh/h

Demand Flow	PL Closed	Unassisted PL	DEM	DLM
250	240	239	239	239
500	502	501	500	503
750	744	740	743	745
1000	993	989	990	993
1250	1241	1238	1239	1245
1400	1387	1363	1385	1389
1500	1483	1370	1479	1464
1600	1532	1392	1537	1484
1700	1550	1362	1549	1519
1800	1555	1333	1552	1519

5.3.3 Speeds

Traffic stream average speeds were also compared as shown in Figures 2a to 2d below. Speeds immediately before and immediately after the passing lane were plotted against the demand volume. .



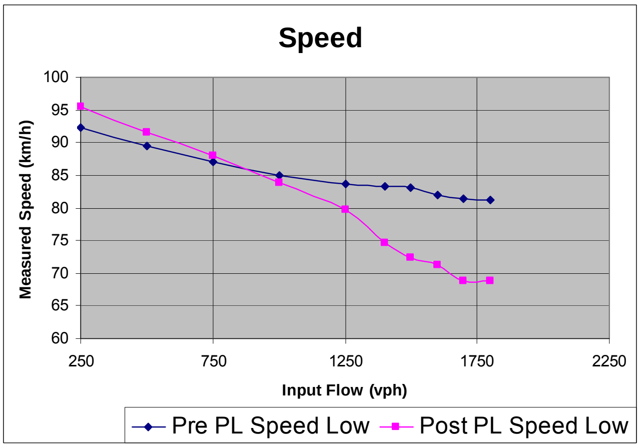
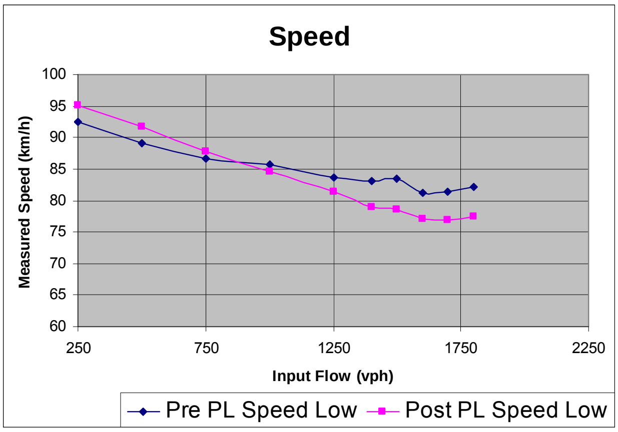


Figure 2a – Closed PL, Mean Speed

Figure 2b – Unassisted PL, Mean Speed

In Figure 2a, the closed or no passing lane case, speeds were found to reduce gradually from the desired speeds for the mix of heavy and light vehicles to around 80 km/h at the capacity flow rate. The difference in speeds between the two graphs reflected the increased platooning and interference between vehicles of different speeds over the 1.45 km between the two measurement points. In Figure 2b, for unassisted passing lanes, the speed downstream of the passing lane fell below that upstream at a demand flow of around 1200 veh/h.



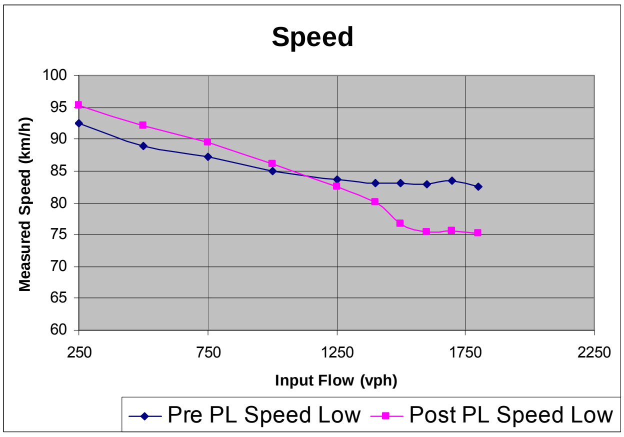


Figure 2c – ITS Early Merge, Mean Speed **Figure 2d – ITS Late Merge, Mean Speed**

For the DEM case Figure 2c, the upstream speed profile against volume was the same as for the passing lane closed (Figuret 2a), but the speed after the passing lane, which was higher than that before passing the lane at low flows (due to release of platooning) became slower above 1,000 veh/h one-way and bottomed out around 77 km/h at capacity compared with 80 km/h for the passing lane closed case (even though the throughput capacities were similar).

For the DLM case Figure 2d, there was a greater speed reduction down to around 75 km/h after the passing lane. The point at which the speed dropped below that upstream of the PL was around 1,100 veh/h one-way, a higher flow rate than for the early merge with the drop-off in speed after that point being steeper. However this may have been be partly due to statistical variation in the simulation results.

This comparison of point speeds must be considered in the context of overall travel time savings, which is discussed in the next section. When considering the upper flow rate at which a DEM layout is used before closing the passing lane, the cross-over point appears to be at aroud 1,500 veh/h one-way. As later discussed, an ITS-assisted strategy that changes from DEM to closure of the passing lane at this flow volume should then give a speed profile against volume as shown in Figure 2a for flows above 1,500 veh/h.

5.3.4 Travel Times

The economic comparison of different merge strategies will depend on the costs of implementation and the benefits, which will be largely the differences in travel time between the options. The following Table 5 compares the savings in travel time per vehicle against an unassisted passing lane. Travel time is measured over between the point immediately upstream of the passing lane to the end of the simulation so as to capture the full extent of the effects (5 km downstream of the PL).

The simulation indicated that either closing the passing lane at flows above 1,250 veh/h one-way or using a dynamic early merge should result in net travel time savings varying from about -0.04 minutes/ vehicle at 1,200 veh/h one-way to 2.5 minutes at 1,800 veh/h one-way. The simulation showed the late merge to give about 2/3 of the savings of the early merge. There was no band of flows where the late merge was superior to the early merge. The advantage of closing the passing lane entirely only occurred at relatively high flow rates. Of the four strategies, the early merge gave the best reduction in travel time over the range of low to high flow.

Table 5 – Base Case, Comparison of Travel Time Savings with Unassisted PL, min

Demand Flow	PL Closed	PL Unassisted	DEM	DLM
250	-0.19	0	-0.02	0.00
500	-0.20	0	-0.04	0.02
750	-0.20	0	-0.04	0.02
1000	-0.17	0	-0.04	0.02
1250	-0.16	0	-0.04	0.00
1400	-0.03	0	0.09	0.11
1500	0.78	0	0.91	0.61
1600	0.91	0	0.98	0.49
1700	1.86	0	1.96	1.46
1800	2.41	0	2.53	1.86

5.3.5 Queue Lengths

When the demand flow exceeds the capacity of the merge, vehicles will form a queue which will lengthen until the demand flow falls below the merge capacity, at which point the queue will start to dissipate. The capacity of the two lane section to hold queued vehicles can be calculated simply as the difference in the demand and output flow rates multiplied by the average vehicle distance headway. The distance headway will be a function of speed, and using the calibrated time headway of 1.45 seconds the distance headway will be 30 to 40m from 75 to 100 km/h.

So a 1,250 m passing lane, assuming full usage of both lanes could, at 75 km/h, theoretically hold 1,250 x 2 /30 = 83 vehicles. The time taken to develop a queue of this length will depend on the difference in the demand and throughput flows. For the DLM layout, the throughput flow was modelled at around 1,500 veh/h. So a demand flow of 1,750 veh/h would completely fill the storage in a 1,250m passing lane in about 20 minutes or, at a higher speed, this capacity would be reached in 15 minutes. An early merge layout would queue back more quickly because of the less even division of flow between the two lanes.

This simple calculation has been compared with the simulation in operation and found to agree.

5.4 Effects of Varying the Input Conditions

A number of tests were made of varying several of the input conditions, one at a time, and observing the effects on the simulation model output for each of the four layout options. The tests were run at a demand flow of 1,500 veh/h one-way.

5.4.1 Varying the Passing Lane Length

The base case was tested for shorter (1,000 m) and longer (1,500m) passing lanes. The change in flows and speeds downstream of the passing lane and travel time per vehicle over the measured section were as shown in

Table 6.

The effect on the measured indicators of changing the length of the passing sections between 1000 and 1500m was insignificant and not detectable by the simulation.

Table 6 – Sensitivity to PL Length, 1500 veh/h

Parameter	PL Closed	PL Unassisted	DEM	DLM
Flow rate after PL, veh/h				
1500m	1486	1338	1481	1464
1250m (base)	1483	1372	1483	1466
1000m	1481	1344	1479	1465
Speed after PL, km/h				
1500m	81.0	71.0	78.0	77.0
1250m (base)	80.0	72.5	79.0	77.0
1000m	80.0	71.0	79.0	78.0
Travel time, min (adjusted for change in length)				
1500m	5.1	6.3	4.9	5.3
1250m (base)	5.1	6.5	5.0	5.3
1000m	5.1	6.5	5.0	5.4

5.4.2 Varying the Percentage of Heavy Vehicles

The percentage of heavy vehicles has been found to be one of the most significant factors of variation in the operation of ITS merge strategies in overseas research. The base simulation was tested for the effect of increasing the HCV% from 10% to 20%. **Error! Reference source not found.** compares flow rate after the passing length, speed at the same point and travel time over the measured section for a change in heavy vehicle composition from 10% to 20%. Compared with an unassisted passing lane, all three of the alternatives show a greater percentage improvement at the higher HCV composition.

Table 7 – Sensitivity to % Heavy Vehicles, % change in Simulation Outputs

Parameter	PL Closed	PL Unassisted	DEM	DLM
increase in HCVs from 10% to 20% (at 1500 veh/h)				
Flow rate after PL, veh/h				
10% HCV	1483 (+101)	1372 (0)	1479 (+107)	1466 (+94)
20% HCV	1459 (+189)	1268 (0)	1464 (+194)	1416 (+148)
change	-1.6%	-7.6%	-1.0%	-3.4%
Speed after PL, km/h				
10% HCV	79.9 (+7.4)	72.5 (0)	78.6 (+6.1)	76.7 (+4.2)
20% HCV	78.0 (+14.3)	63.7 (0)	72.9 (+9.2)	68.9 (+5.2)
change	-2.3%	-12.1%	-7.3%	-10.2%
Travel time, min				
10% HCV	5.1 (+1.4)	6.5 (0)	5.0 (+1.5)	5.3 (+1.5)
20% HCV	5.2 (+2.2)	7.4 (0)	5.1 (+2.3)	5.8 (+1.6)

Parameter	PL Closed	PL Unassisted	DEM	DLM
change	+2.6%	+14.1%	+3.5%	+8.8%

This improvement is in agreement with the overseas research findings that ITS merge strategies are more effective if there is a high proportion of heavy vehicles. This can be explained by an ITS-assisted merging strategy helping to provide better gap selection, especially at locations where traffic streams can be easily disrupted by stop-start behaviour, as vehicles must adjust their merging speeds to provide larger critical gaps for heavy vehicles compared to the gaps needed for passenger cars.

While the speed after the passing lane dropped off with increasing HCV% in the simulation, the overall travel time was only marginally affected and was similar to the closed passing lane case, the speed drop off being more than compensated by gains elsewhere along and following the passing lane. This marginal difference in travel time savings applied to the modelled case of a 5 km downstream effective length rather than the 1.5-2 km downstream effective length of a 2+1 layout.

5.4.3 Varying the Platooning before the Passing Lane

The degree of platooning, measured as the percentage of vehicles following within 4 seconds of the preceding vehicle, is controlled by the length of single lane section in advance of the passing lanes. The main simulation was run with a 5,000m advance length and this was tested for sensitivity to 3,000m and 1,000m which has the effect of reducing the time and distance over which platoons can form from the initial randomly distributed flow.

For a demand flow of 1,500 veh/h one-way, the percentage of following vehicles ahead of the two-lane section varied only marginally with a reduction in the advance length from 94.8% following at 5,000m advance length to 93.7% at 1,000m advance length. The sensitivity of merge capacity, and speed downstream of the PL to the advance length was relatively small as shown in Table 8 below.

A similar analysis was run for 500 veh/h demand flow to see whether there was any increased sensitivity to the advance single lane length/ percentage following. The percentage following reduced to 70%, 60.5% and 46.6% respectively for 5000, 3000 and 1000m advance length. At this lower demand flow the performance of all options was similar and statistically indistinguishable.

The conclusion was that the degree of platooning on entering the passing section, in the range of flows where ITS-assisted merging is potentially effective, is going to be very high whatever the passing opportunities in advance of the passing section. Consequently the length of road in advance of the passing lane in the range of 1,000m to 5,000m tested will have little effect on the results. The benefits of an ITS-merge strategy should then be only marginally less for a 2+1 layout at, say, 2,500m spacing, than for passing lanes at 5,000m spacing.

Table 8 – Sensitivity to Percentage Following at 1500 veh/h

Parameter	PL Closed	PL Unassisted	DEM	DLM
Single lane section length and % following upstream of 2 lane section				
Flow rate after PL, veh/h				
5,000m/94.8%	1483 (+113)	1370 (0)	1479 (+109)	1466 (+94)
3,000m/94.6%	1496 (+156)	1340 (0)	1492 (+152)	1473 (+133)
1,000m/93.7%	1498 (+189)	1309 (0)	1495 (+186)	1478 (+169)
Speed after PL, km/h				
5,000m/94.8%	79.9 (+7.4)	72.5 (0)	78.6 (+6.1)	76.7 (+4.2)
3,000m/94.6%	80.7 (+10.3)	70.4 (0)	78.9 (+8.5)	77.7 (+7.3)

Parameter	PL Closed	PL Unassisted	DEM	DLM
1,000m/93.7%	81.7 (+12.5)	69.2 (0)	80.4 (+11.2)	77.0 (+7.8)
Travel time, min				
5,000m/94.8%	5.1 (+1.4)	6.5 (0)	5.0 (+1.5)	5.3 (+1.2)
3,000m/94.6%	5.1 (+1.4)	6.5 (0)	5.0 (+1.5)	5.3 (+1.2)
1,000m/93.7%	5.1 (+2.1)	7.2 (0)	4.9 (+2.3)	5.2 (+2.0)

5.4.4 Location of Merge Signing for DEM

The dynamic early merge option was tested for a variation in the location of the first merge sign, from the base case of 500m before the end of PL to 250m and 750m. Tests were carried out at a demand flow of 1,500 veh/h one-way.

Table 9 – Sensitivity of DEM to Location of Merge Signing, 1500 veh/h flow

Parameter	250m	500m (base)	750m
Values			
Flow rate after PL, veh/h	1424	1479	1482
Speed after PL, km/h	74	79	79
Travel time, min	5.5	5.0	5.0
Percentages			
Flow rate after PL, veh/h	-3.7%	0	+0.2%
Speed after PL, km/h	-6.3%	0	0%
Travel time, min	+10%	0	0%

There was no difference in performance between 500m and 750m positioning of the first ITS merge sign. However, if moved to 250m from the end of the passing lane, the performance deteriorated, the capacity reducing by 3.7%, the speed reducing by 6.3% and the travel time increasing by 10%. The effect of moving the merge sign further and further from the merge point is to limit the length available for overtaking and in the extreme would turn the facility into a single lane operation.

5.4.5 Comparison of European (Fast Merge) and NZ (Slow Merge) Lane Merge Markings

The NZ lane markings were the base case and involve the slower kerbside lane deviating right into the faster (overtaking) traffic lane at the end of the merge. The European marking involved the kerbside slower traffic not deviating at the end of the merge while the fast overtaking lane is required to move left into the slow lane. These have been called "slow merge" and "fast merge".

For the DEM option, the difference in performance between the slow and fast merge was almost too small to be identifiable and was only fractions of one percent for both capacity and speed downstream of the passing section. It appeared from the simulation that the fast merge was fractionally superior.

For the DLM option, operating without any speed restrictions, the slow merge appeared to have a slight capacity advantage at very high demand flow (1,700 veh/h one-way), although there was no difference in speed downstream of the passing lane. However over the range of flows up to 1,600 veh/h one-way there was no apparent difference. The results at very high flow could be a result of statistical variation between simulations.

5.4.6 Effect of Speed Restriction on the Dynamic Late Merge Option

The late merge option (slow merge, NZ markings) was specified without any speed restriction in the base case. Speed reductions were those occurring through the car following model in the simulation rather than any imposed limits, other than the open road 100 km/h. Tests were run with speed restrictions applied to the passing lane section following the first sign advising of the late merge. Sensitivity tests were run at 1,500 veh/h one-way demand flow and for 70 and 50 km/h speed restrictions.

Table 10 – Sensitivity of DLM to Speed Restriction

Parameter	100 km/h (Base)	70 km/h	50 km/h
Values			
Flow rate after PL, veh/h	1466	1471	1449
Speed after PL, km/h	77	74	73
Travel time, min	5.3	5.3	5.8
Percentages			
Flow rate after PL, veh/h	0	+0.3%	-1.2%
Speed after PL, km/h	0	-3.9%	-5.2%
Travel time, min	0	-0.4%	-9.6%

The sensitivity test was also run for the Euro (fast merge) markings, and gave closely similar results. The simulation did not show any benefit from applying a speed restriction of 70 km/h and an apparent disbenefit for a restriction of 50 km/h. However, the simulation indicated that a speed restriction of 70 km/h could be applied without operational disadvantage to the DLM and may be regarded as advisable for general traffic safety and as a check against overly aggressive behaviour – matters that cannot be readily modelled in the VISSIM simulation.

5.5 Conclusions from the Simulation Modelling

5.5.1 Main Findings

The VISIM simulation modelling gave results that were generally consistent with the overseas research findings. The main points from the modelling are summarised below. It should be borne in mind that simulation results do not take into account all of the potential human behavioural factors, so these simulation results should be regarded as a starting point only for further investigation.

- The greatest throughput capacity was achieved by closing the passing lane at high flow rates – this is because the single lane downstream of the passing lane governs capacity, and the capacity of the merge acts as a bottleneck that cannot exceed the downstream flow rate;
- However, closing the passing lane at a directional flow below 1,350 veh/h one-way was found to be inefficient, as an unassisted passing lane is operationally superior to having no passing lane at all up to this rate of flow;
- The dynamic early merge, with the car following model calibrated as closely as possible to observed headway distributions, gave better merge behaviour at high rates of flow than did an unassisted passing lane and appeared to slightly exceed the performance of closing the passing lane at flows between 1,350 and 1,500 veh/h one-way. The reason why this could be possible is the passing that has taken place over the two lane section, which marginally improves the average travel time of all vehicles over the measured section. Given that closing a passing lane requires deployment of some temporary traffic control measures (temporary signage and coning off lanes etc), that would have to be put in place for a period of time and so operate at low flow as well as high flow, an ITS-assisted DEM would appear to give the benefits of more efficient handling of flows in the 1,350 to 1,500 veh/h range without losing the benefits of handling lower flows;
- The dynamic late merge strategy as modelled gave poorer rather than better performance at high levels of flow than the dynamic early merge; the only advantage of dynamic late merge was that it provides more storage length in the two-lane section for traffic queues

when the capacity of the merge and single lane exceeds the upstream demand flow, typically at flow rates about 1,700 veh/h;

5.5.2 Sensitivity to Parameters

The findings from the sensitivity tests were as follows:

- **Percentage Heavy/Slow Vehicles**: the results were most sensitive to the proportion of heavy (as by extension, slow) vehicles; the more variability there is in vehicle desired speed and acceleration capacity within the traffic stream, the greater the relative advantage of both unassisted passing lanes and ITS-merge assisted arrangements;
- **Location of the ITS merge signage**: for dynamic early merge, reducing the distance between the merge sign and the end of the passing section to below 500m appeared to reduce the effectiveness of the DEM; increasing it above 500m on the other hand did not significantly improve the performance of the arrangement and started to cut into the available passing length;
- **Upstream passing opportunities**: the conditions upstream of the passing lane, in terms of available passing opportunities did not have a significant effect on the performance of the ITS merge options; this was because of the high flow rates at which the ITS merge is useful and at these flow rates the traffic stream becomes heavily platooned within a relatively short upstream length. So the percentage of following vehicles coming into the passing lane at flow rates greater than 1,350 veh/h exceeds 90%;
- **Length of passing section**: varying the length of the passing section between 1,000, 1,250 and 1,500m had no significant effect on the relative performance of the ITS merge options in the simulation; again this was most likely due to the relatively high flows at which the ITS-merge layouts become effective.
- **NZ/European Merge Markings (Slow/Fast Merge**): very small performance advantages were detected for the European (fast) merge compared to the NZ (slow) merge which is in the direction that one would expect; however we qualify this with the observation that this result could be influenced by how well the VISSIM model is able to reproduce merge behaviour and some of the VISSIM calibration parameters. Simulation modelling alone is probably insufficient to prove this finding one way or the other.
- **Adding Speed Restriction to DLM:** the option of adding either a 70 km/h speed restriction 500m before the end of the passing section did not have any effect on the simulation performance compared with the open 100km/h limit, with traffic speeds controlled by driver observation of the traffic stream. A 50 km/h limit gave a small reduction in the performance of the DLM. In itself this is not a reason to discount a speed restriction sign, which could still be useful for signalling conditions to drivers, calming aggressive behaviour and potentially leading to safer operation. This result does indicate that there is possibly little performance advantage from adding the restriction sign.

6 Indicative Economic Analysis and Scope for Application

6.1 Economic Analysis Method

The costs of an ITS-assisted merge installation will potentially comprise: (i) the additional costs of an ITS layout over and above that of a conventional unassisted passing lane or 2+1 layout; (ii) cost savings from deferring four-laning of road sections that are approaching capacity; (ii) operational savings by not having to manually close the passing lanes.

The road user benefits are potentially: (i) reduction in travel time and congestion between the DEM installation and passing lane closure over the 1200 to 1500 veh/h one-way flow range; (ii) any associated vehicle operating cost differences; and (iii) any net safety differences between the non-ITS and ITS layout, based on increased or decreased crash risk and severity.

For purposes of comparing the performance of different layouts under flow rates less than the maximum capacity, the main output statistic is the average time per vehicle through the simulation. This does not differentiate between congested and uncongested travel time, but we can assume that as we are operating at high flow rates, all travel can be regarded as “congested” in the meaning of the EEM (V/C > 70%). Consequently for BCR purposes we can use differences in the average time per vehicle as the main benefit input applying the full congestion (CRV) addition.

There will be some vehicle operating cost effects due to speed changes, but these are typically very difficult to estimate and are relatively small in comparison to time cost. We could make a nominal 10% addition to the time cost in recognition of VOCs but this will not significantly affect the result bearing in mind other approximations.

Differences in crash rates and costs are unknown and, assuming that a practical installation would be safety audited, should not result in any reduction in safety performance. Safety implications will to some extent be defined from subsequent driver simulation and could potentially be a constraint on how ITS merging can be implemented.

The road user benefits are therefore calculated from the difference in annual vehicle-hours of travel time between an unassisted passing lane, and the ITS-merge options (and also the option of closing the passing lane at 1200 vph/h one-way).

6.2 Indicative Capital and Operating Costs

The cost of providing VMS signage will depend on location and the extent of buried services required. We have based this rough order of cost on a supply cost per VMS sign of $25,000 plus services and installation on-cost of 100% and ongoing maintenance and operational costs of 5% of the capital cost per annum over a 15 year service life, which has been taken as the time horizon for the analysis. For a gated layout, the signs are doubled up one each side of the road within the passing lane section. For an overhead gantry, the cost per gantry is estimated at roughly $250,000, but because of the high cost of such an installation, this option has not been carried through to the economic analysis.

The DEM/DLM layouts shown in Figures 1 and 2 have been used as the basis for the cost comparison and the simplified layout suggested by NZTA has also been costed. A comparison of the option costs is shown in

Table 11.

Table 11 – Cost Comparison of ITS-Assisted Merge Layouts, $ million

	single side	**gated**	gantry	**NZTA option** gated
No. of Signs				
DEM	4	7	2 + 2G	10
DEM/DLM	6	10	3 + 2G	-
Capital Cost, $m				
DEM	0.20	0.35	0.60	0.50
DEM/DLM	0.30	0.50	0.65	-
Including 5% per annum O&M, discounted over 15y at 8% dr				
DEM	0.29	0.51	0.87	0.36
DEM/DLM	0.43	0.72	0.94	-

Note: numbers and costs are extra over the numbers and costs of a conventional passing lane

6.3 Cost Savings from Deferred Construction

Once an existing passing lane or 2+1 layout exceeds the hourly capacity of an ITS installation, the next step will be to move to four-laning in the peak direction and probably for the road as a whole if the directional peaks are similar. Once the peak hourly flow reaches 1,200 veh/h one-way, reducing the spacing of passing lanes will only act to add marginally to passing opportunities and, as has been shown, the degree of platooning on the one-lane section will remain very high. In the case of 1.5km passing lanes at 5km intervals, moving directly to four-laning involves adding 3.5 lane-km in a 5 km section. For a 2+1 layout, it involves adding 2.5 lane-km in a 5km section.

The cost savings from deferring lane construction depend upon the construction cost, which can vary from $300,000 to over $1.00 million per lane kilometre, using NZTA's Economic Evaluation Manual average values by terrain. On the urban fringe where the value of land is high, this will add to the cost. The cost of passing lane construction will be very particular to local circumstances of the project and could be higher or lower than this range.

Annual traffic growth rates are typically in the range of 1.5% to 2.5% per annum (linear). The decision on when to upgrade will depend on the hourly traffic flow profile over the year. If upgrading is done before the highest peak directional hourly flow reaches 1,200 veh/h, then the benefits from travel time saving will be negligible in the first year. If upgrading is deferred for too long, then there is a road user cost in passing lane closure at the high peaks.

In the following analysis, we have used an hourly flow and directional profile from Cenek & Lester (2008) (Table 6) which gives flow profiles for rural non-recreational strategic routes and for rural/urban fringe routes. We have timed the upgrade to four lanes when 1,200 veh/h one-way is exceeded for more than 125 hours or more in a year, corresponding to flows on holiday weekends. This is equivalent to an AADT of 17,000-21,000 veh/day, depending on directional split and peak hour flows. In the ITS-assisted case, we have assumed that upgrading to four lanes would be when the peak flow exceeds 1,500 veh/h one-way for 125 hours per year in the first year.

The years that the upgrade is deferred is then the difference in hourly peak flow of 300 veh/h one-way divided by the annual increase, as shown in

Table 12.

Table 12 – Years that 4-Laning is Deferred

Traffic Growth % linear	Rural Strategic non-recreational	Urban/Rural Fringe
1.5	21	26
2.0	16	20
2.5	13	16

6.4 Cost Savings in Passing Lane Closures

The indicative costs of manually closing passing lanes at high weekend peaks provided by NZTA based on three Wellington region passing lanes, is of the order of $2,500 to $3,000 per PL over a weekend, say for 10 hours over which the 1,200 veh/h is exceeded. The extent of cost savings will depend on the point at which the passing lane is upgraded, either to ITS or four laned. Based on the 50 hours exceeded per year, used above, this would apply to 5 such weekends or $12,500 to $15,000 per year. These savings would apply annually, more or less, up to the point that the road was eventually four-laned, that is over the years that the ITS installation defers the need to four lane.

6.5 Travel Time and Congestion Benefits

From the simulation, the travel time saving potential in the band of directional flow between 1,200 and 1,500 veh/h is about of 0.32 minutes per vehicle (average of -0.04, 0.09 and 0.91) compared with a closed passing lane of 0.20 minutes per vehicle (average of -0.16, -0.03, 0.78). The net saving of 0.12 minutes per vehicle is relatively even across this flow range.

The evaluation of time and congestion benefits has projected the number of hours per year where directional flow is above 1,200 veh/h and above 1,500 veh/h. The average hours/year in the band 1,200 to 1,500 can be approximated as half the difference between these two projections. The annual growth rate in traffic is applied to the AADT and the number of hours per year above 1,200 and 1,500 veh/h looked up from a profile of the hourly flow. The upper limit of benefit is when there are more than 50 hours/year above 1,500 veh/h, at which point it is assumed full four-laning would occur.

The estimate of discounted project benefits varies between $83,000 and $91,000 over the 15 year evaluation period depending on traffic growth (lower growth gives higher benefits because the ITS facility is able to handle the flows over a longer period until four-laning is required). The road user benefits appear to be a relatively small component of the overall savings, the majority coming from deferral of construction and from elimination of manual intervention to close the passing lane at peak periods.

6.6 Overall Evaluation

The overall evaluation indicates that the capital and operating costs of an ITS installation to be less than the cost savings from deferring upgrading to four lanes together with the savings in managing passing lane closures manually, over a range of traffic growth rates (1.5%, 2.0%, 2.5%) and cost per lane kilometre for four-laning ($0.30 million to $1.0 million). The present value of cost savings varied from near zero for flat terrain and high traffic growth, up to $2.5 million for mountainous terrain and low traffic growth.

The travel time and congestion savings to road users were estimated to be relatively modest by comparison at $0.09 million. It appears that the potential economic advantage of ITS-assisted merging lies in the savings to the roading authority in being able to operate passing lanes and 2+1 layouts up to a higher flow (from 1,200 to 1,500 veh/h one-way) so that upgrading costs are deferred for 10 to 20 years and manual management of lane closures in high peaks are substituted with the ITS system.

Circumstances favourable to the economics of an ITS-assisted installation are:

- Relatively high cost of land and construction for adding or extending an auxiliary lane, such as in difficult terrain and geology and/or where land values are high on the urban periphery;
- Traffic flow at or approaching 1200 veh/h one-way for 150 or more hours per year, in the region of 13,500 vehicles/day AADT, depending on the shape of the annual hourly flow profile;
- Relatively low traffic growth rate, which has the effect on increasing the time that four-laning can be deferred by ITS-management of passing lane capacity.

While the results of this rough order of cost and economic evaluation appear to be positive, they of course rely on the performance of the ITS-assisted layout being confirmed through subsequent driver simulation and field trials.

6.7 Potential Market for ITS-Assisted Merging

NZTA has estimated that over the next 25 to 30 years in addition to the existing 200 km in the range of 10,000-25,000 vpd a further 900 km approx of rural state highway currently under 10,000 veh/day will move into this higher daily flow range, demanding more than a two-lane road with periodic passing lanes. Assuming that about 550 km of those rural state highways with projected 10,000 to 25,000 vpd would have one-way peak hour traffic flows greater than 1,200 veh/h and based on 1.5 km passing lanes at typically 5 km spacings, about 160 passing lanes would require regular lane closure over the next 30 years. If these rural state highway sections were developed as 2+1 lanes without assisted merging, there would be about 330 passing lanes that would have to be closed on a regular basis. A full 2+1 layout would potentially defer the need for four-laning, particularly if the upper limit of operation of 2+1 could be extended by the equivalent of 300 veh/h in the peak direction as indicated in the modelling.

Overall, if there is say 1,100 km of 2+1 lane roads in the future where ITS would defer the need for four-laning by 10 years or more, with a cost differential of $2 million per km assuming treatment in both directions, this amounts to a potential discounted cost saving in excess of $1,200 million. In addition, there could also be a number of isolated passing lanes where seasonal peaks may warrant either a permanent or temporary ITS treatment.

7 Proposed Ongoing Programme and Staging

7.1 NZTA Proposed Staging and Scope

The ongoing research programme is envisaged as follows:

Stage 2:

(i) Cognitive testing using driving simulation

(ii) **Stage 2 Report** on the driver simulation, including recommendation and specification and costing of field trials as a third stage

Stage 3:

(iii) Field trials which may include preparatory off-road testing in a controlled situation prior to testing on the public road or trial on existing four-lane state highway with one direction of the two lanes merged into one lane.

(iv) Stakeholder consultation and regulatory requirements definition

(v) **Stage 3 Report** with recommendations on implementation

7.2 Stage 2 - Driver Response Testing

7.2.1 General

The microsimulation modelling necessarily makes the basic assumption that drivers perceive, comprehend and respond appropriately to the ITS signage. These behavioural responses are not built into the modelling to any significant extent and several different arrangements of signs could direct a similar action from the driver but be perceived and responded to differently.

The first assumption of perception concerns the visibility of the VMS sign taking account of road geometry, surrounding traffic which could partially obscure the signage, speed of travel, lighting conditions, and other calls on driver attention. The positioning, size and clarity of the signage is important here.

The second assumption of comprehension is important, particularly in regard to the message conveyed. Messages need to be short and clear. Where a message sign is mounted with an arrow or speed restriction sign, then the designer needs to be sure that both are taken in and comprehended by the driver.

The third assumption is driver response once the message is perceived and understood. Messages to keep left, to merge, or to take turns in a zip merge, may only be partially complied with for many reasons, including level of aggression and jockeying for positional advantage, tendency to maintain lane position, to occupy the outer lane. Faced with unfamiliar signage drivers may change their normal behaviour, such as by slowing down more than required.

For these reasons a stage of cognitive testing is proposed as the next stage of research into ITS-assisted merging.

7.2.2 Form of ITS-Merge for Study

From the results of Stage 1, it appears that a dynamic early merge (DEM) arrangement offers the best opportunity for extending the capacity flow for a passing lane or 2+1 lane, by about 300 veh/h from around 1200 to 1500 veh/h. The dynamic late merge (DLM) did not perform as well in the simulation. However, the overseas research for highway construction zones has given a more positive assessment of DLM and it would be possible to design VMS signage that can be adapted to either situation.

We propose that both forms be incorporated into the driver response testing, based on the arrangements shown in Figure 3 and Figure 4 or on NZTA's simplified arrangement. The form of messages, use of messages versus arrows, single sided versus gated or gantry arrangement would be incorporated into the driver simulation tests.

7.2.3 Driver Simulation Test Facilities

In order to perform the driver response testing, a driving simulator offers a safe controlled environment where the reactions of drivers can be scientifically monitored and questioned to give their subjective views of ITS layouts. The disadvantage of simulator testing is that the environment, while made as realistic as possible, is nevertheless artificial although it may be based on actual visual imagery of a real length of road. Using video of a specific section or sections of road with incorporated computer graphic imagery for such trials is desirable as it will allow possibly unexpected aspects of the visual field to be included in the simulation. This could be one of the passing sections that has been regularly closed in holiday peaks if the geometric characteristics are suitable.

We have discussed the problem in general terms with Dr Charlton of the Waikato Traffic and Road Safety (TARS) Research Group, which is the only such test facility in New Zealand and the most suitable group to carry out the necessary research into driver behaviour.

7.2.4 Timescale and Indicative Cost

From discussions with TARS, we are advised that a 2-3 month lead time would be required to develop the test scenarios prior to starting the experimental work with recruited test subjects. Also, the driver simulator is a unique facility with a forward programme of bookings and this project would need to be pre-programmed. For planning purposes, we suggest that a 3 month lead time from agreement to proceed with Stage 2 to carry out initial planning of the driver simulation research and programming for access to the simulator, a further three months to set up the test scenarios, a month for the simulator testing and a further month for analysis and reporting of results, a total of 8 months.

We have been advised that typical costs for the driver simulation study based on previous similar work are advised to be in the order of $35,000, depending on the eventual design, but would need to be worked up in detail. We suggest that a reasonable contingency should be allowed for, as we expect that this work will be at the more demanding end of such studies, as it involves multiple factors over about a 2 km length of road and may require several test scenarios. We suggest an initial indicative budget of $50,000.

7.2.5 Simulation Methods

Two simulation methods are available. The first is to use video as a point of reference while building a completely CGI scenario. This method gives the most control over the behaviour of the other traffic and is the first choice as driver performance in response to the signs is the primary data of interest. The second method is to use video and introduce new signage into that video using CGI "special effects". This latter approach produces the most photo-realistic simulation, but the lack of control over the simulated vehicles means that this method would probably be best-suited if driver detection and comprehension of the signs were the primary data of interest (as opposed to driver speeds and lane changing).

We understand that for previous work, TARS has created the 3D scenario using RGDAS road alignment data, using an actual road section as the basis for the simulation. The CGI scenery can best be generated using a video of the actual road section. The ITS signage would then need to be built as CGI elements to fit into the scenario. The resources to do this would depend on degree of complexity and whether there are any prior CGI elements that can be adapted or, more likely, whether these would need to be created from scratch. Most of the cost lies in the set-up of the simulation, with a relatively lesser marginal cost of testing

The nature of the simulation experiments would involve recruitment of drivers, pre-screening to select those who can adapt to the simulated driving environment, practice time of ½ hour and then experimental time typically of 1 hour.

The aim of the experiments would be to monitor the drivers' control of the vehicle in response to the road and traffic environment as presented, including the ITS signage (as opposed to asking the driver questions about perceptions and understanding of the signage although this could be added as a post-simulation activity). It should be noted that while the trajectory of the driven vehicle in the traffic stream can be varied, there is only limited ability to model the reaction of the other computer simulated traffic in response. So, the simulated traffic stream appears as a fixed element to which the driven vehicle has to adapt. How well this would work in the case of the late merge at high flow rates would need to be determined. Probably, the simulated traffic stream would have to anticipate the behaviour at the merge in regard to leaving a gap versus blocking the merging flow. The nature of this behaviour would have to be developed from some empirical work on behaviour at forced merges and compliance with zip-merge signage. No provision is made for studying this behaviour within this research program; it has been assumed that some guidance is available from the literature.

7.2.6 Stage 2 Outcomes and Actions

The Stage 2 testing would focus on deciding the design and layout of an ITS DEM installation, with capability for adaptation to DLM. It would make conclusions on the use of single sided versus gated versus gantry arrangement of VMS signs. At the end of this stage we propose that a specimen design of the preferred arrangement be specified, drawn up and costed, so that the economic benefits can be confirmed, prior to a decision on whether to carry through into Stage 3.

A report covering the requirements needed to permit the project to go forward to road trials would need to be prepared.

Overall, we suggest that 12 months be allowed for Stage 2, and an overall indicative budget of $100,000, broken down as $50,000 for the driver simulation and $50,000 for development of a specimen design, costing, review of economics and preparation of a report conforming to an application for road trials together with a costed programme.

7.3 Stage 3 - Road Trials

The official requirements for road trails are set out in the TCD Manual Appendix A. The rules apply to non-conforming devices. Even if the devices conform to current rules, deployment in a new situation outside established practice should observe these requirements as a precaution.

The application to NZTA for a road trial requires a report setting out the issues, background, technical analysis, impacts and risks assessment, expected safety and efficiency gains, consultation undertaken and proposed and the assessment method proposed. Most of this will have been undertaken in the foregoing research. A pro-forma for the application is given in Appendix AA to the TCD Manual.

The formal procedure is for the application to be put before the Traffic Control Devices Steering Group, which meets quarterly. NZTA's decision then rests on the Steering Group's recommendations, the policy, regulatory and legislative effects of implementing the proposal, communication requirements, resource availability and relative priority against other initiatives.

Clearly these matters will need to be substantially covered in the foregoing research, and the views of the Steering Group should be obtained once the proposals are at a sufficient level of development to avoid any later difficulty with the application.

A Gazette notice will be required if the application contains non-conforming elements, and prior publication on the NZTA website.

If the trial is successful, adoption of the new traffic control devices/arrangement would be subject to any changes required in the Land Transport Rule on Traffic Control Devices which would also require gazetting.

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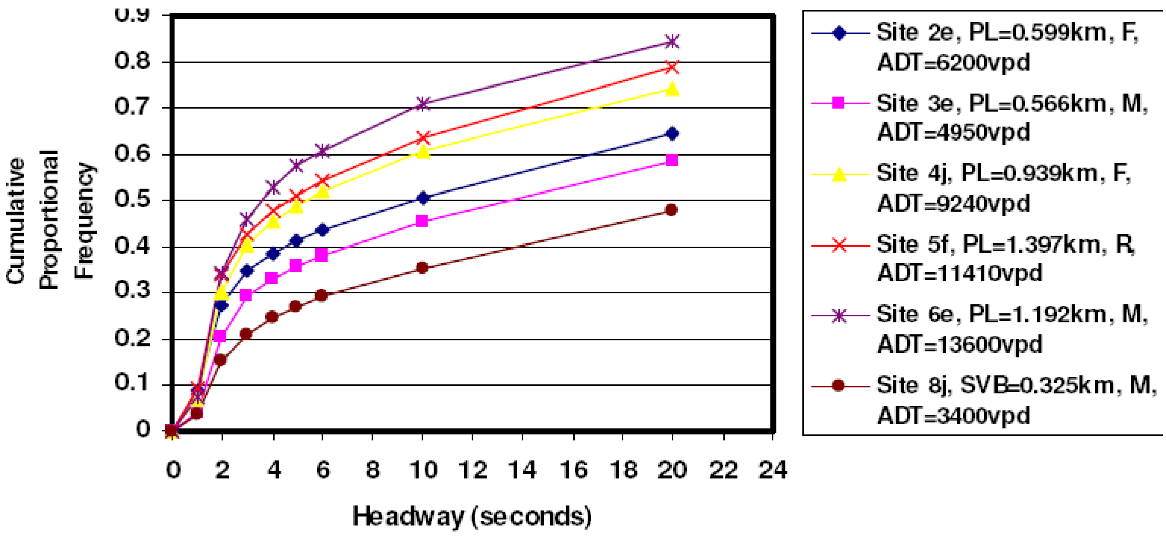
Appendix A – Notes on Model Calibration

A1 Sources

The results of passing lane surveys conducted in New Zealand as reported in Cenek and Lester (2008) and Koorey and Gu (2001) were used in the microsimulation model calibration. Cenek and Lester (2008) conducted surveys on six passing lanes, of which two sites (4j and 5f) were located on gentle slopes and had passing lane lengths (939m and 1397m respectively) similar to those used for this research. The surveyed headway cumulative distribution was adopted as a benchmark for headway distribution calibration, shown in the plot from the report below.

Figure A1 – Headway Cumulative Distribution, from Cenek and Lester (2008)



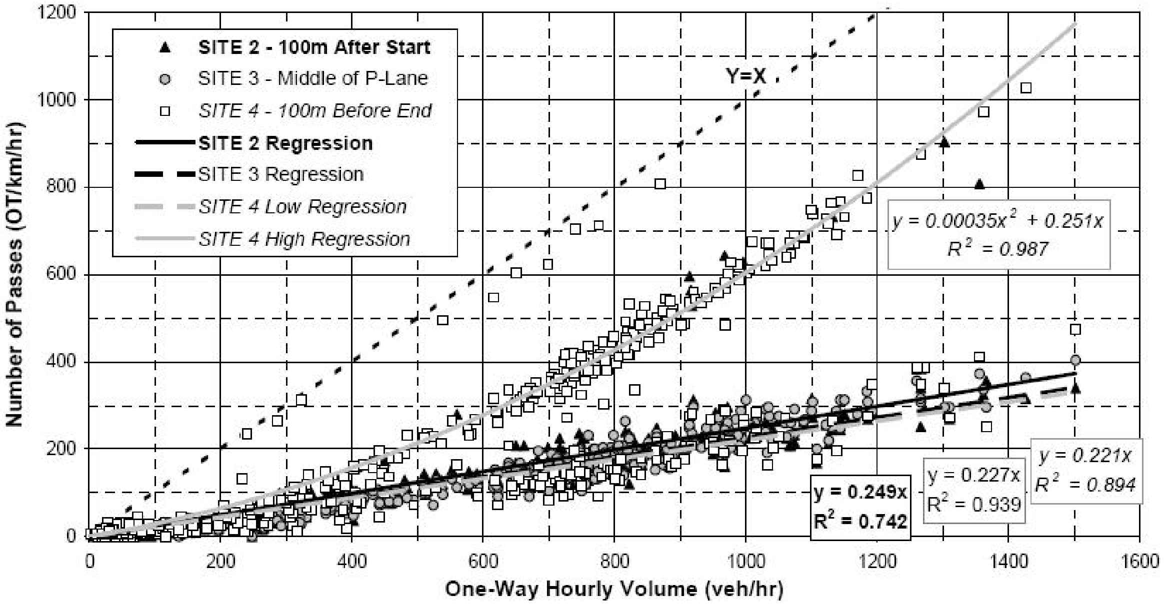
The percentage of vehicle following immediately upstream of the passing lane, and the difference between the percentage following upstream and downstream, were also used as benchmarks for calibrating the model. The headway for being considered a following vehicle was 4.0 seconds. The reported percentages following for the two sites are shown below:

Site	Flow Rate veh/h	No. of hourly readings	Light Vehicles Towing and heavy Vehicles %	Percentage of Following Vehicles (4 seconds criterion)	
				Upstream	Upstream-Downstream
4j	451-500	6	6	56.9	4.2
	301-400	15	11	51.1	5.8
	201-300	13	13	45.8	5.8
5f	400-450	11	18	54.9	4.6
	300-400	20	17	50.3	5.7

Source: Cenek and Lester (2008), Table 9

The other calibration source was Koorey and Gu (2001), who found the following relationships between passing and passing lane directional volume for a 1.0 km length passing lane Otaihanga on State Highway 1.

Figure 3.14 Number of passes observed on Otaihanga passing lane.



Source: Koorey and Gu (20010, Figure 3.14

"*Vehicle counts at Sites 2, 3 and 4 were subtracted from the average count from Sites 1 and 5 to infer the number of vehicles using the passing lane adjacent to Sites 2, 3, and 4. Figure 3.14 shows the quarter-hourly points for these numbers relative to the overall traffic flow, scaled up to give hourly passing rates*." (Koorey and Gu, 2001)

A2 Calibration Methodology

A2.1 Microsimulation Model Setup

The VISSIM model was set up for one directional travel, without opposing traffic. VISSIM and the other microsimulation models considered do not include modelling of passing opportunities in the opposing traffic lanes, so there is no advantage in including opposing traffic in the simulation. It has the effect of an implicit assumption that there are no passing opportunities on the lead-in section of the simulation.

With this in mind, the simulation was set up with the following configuration:

- 3,000m lead-in section of single lane, with the opportunity to increase or reduce the lead-in section to change the platooning pattern at the start of the passing lane
- the two lane overtaking section, initially set at 1,000m including tapers,
- a 2,000 m single lane section downstream of the passing lane.
- 5% heavy commercial vehicles (HCV)
- Five data collection points for microsimulation output data collection were selected:

  #1 - 100 metres before the two lane section starts

#2 - 100 metres after the two lane section starts on the left lane

#3 - mid-way along the two lane section in the left lane

#4 - 100 metres before the merge taper the left lane

#5 - 100 metres downstream of the two lane section

- A passing lane ends/merge sign was set at 200 metres before the merge taper;
- The diverge point was coded in a way such that slow vehicles continue to go straight while overtaking vehicles would need to move into the new lane on the right.
- The merge point was coded so that the slow vehicles move right into the fast lane (slow merge, NZ style passing lanes);
- Models were set up for directional flows between 250 and 1800 veh/h
- Each model was coded to run 5 iterations for an 1-hour simulation time plus 5 minutes warm-up period each; this was extended to 10 iterations where there was a high coefficient of variation in the output parameters
- The simulation resolution was set at 1 time step per simulation second.

A2.2 Model Outputs and Calculations

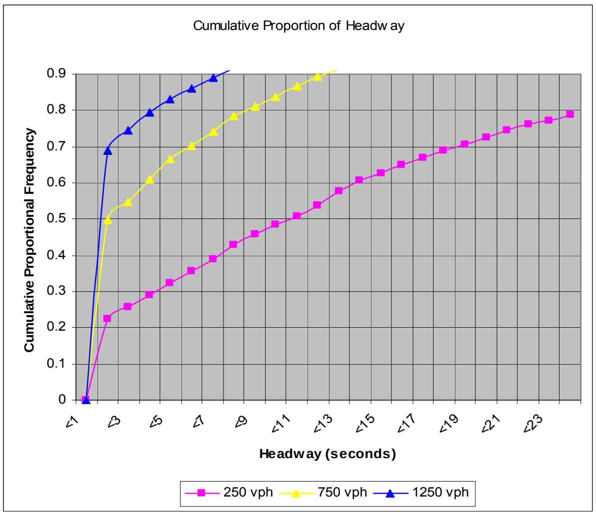
Individual vehicle data was output from the model at each of the five data collection points. The following output indicators were obtained at each flow rate.

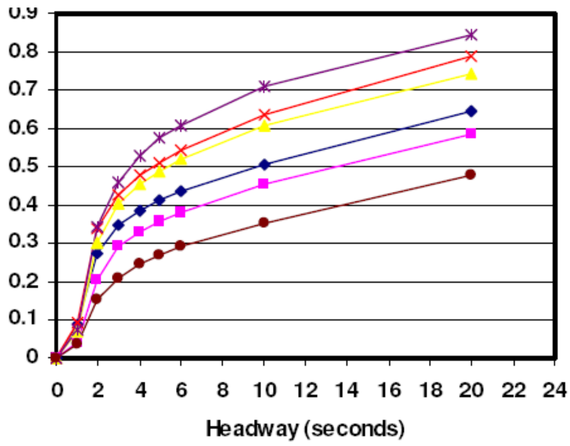
The headway of each vehicle was collected at data point #1, as the difference between the simulation times of the preceding vehicle and the sample vehicle. These headways were then grouped into 25 bins at 1 second intervals and the cumulative distribution plotted similarly to Figure A1. The headway calculation was also performed for point #5. The percentage following was calculated based on a 4 second headway. The passing rate at the three points within the passing lane (data collection points #2, #3 and #4) were calculated as the same way as described in Koorey and Gu (2001). The passing rate for all flows and all iterations were then plotted and a linear trendline with zero intercept was added.

A2.3 Calibration Process

A2.3.1 Initial Model Output Results using Default Parameters

The following figures compare the headway from the model with the VISSIM default values with the results found by Cenek and Lester (2008). The pink and yellow lines in both graphs represent similar flow condition (200~250 and 700~750 veh/h). The modelled headway cumulative distribution has a much higher proportion of vehicles with less than 2 seconds headway at flow 750 and 1250 veh/h. The modelled headway cumulative distributions also increased at a faster rate than the benchmark. In other words, the tails of the modelled cumulative distributions are higher than the benchmark.





% Following Comparison

Modelled Flow	Modelled % Following	Modelled Difference in % Following	Benchmark Flow Rate	Benchmark % Following	Benchmark Difference in % Following
250	29%	0%	201-300	45.8%	5.8%
750	62%	-2%	701-750	68%	4.4%
1250	80%	-2%	-	-	-

The modelled percentage of vehicles following upstream of the passing lane and the difference in percentage following between upstream and downstream of the passing lane were both were generally lower than the benchmark.

Passing Rate Comparison

Data Collection Point	Modelled Passing Rate	Benchmark Passing Rate
2	34%	22%
3	43%	23%
4	87%	≈ 70% at 1250 vph[2]

The modelled passing rates were all higher than the benchmarks. The default VISSIM parameters needed adjustment to better match the field data.

A2.3.2 Model Calibration

The first step was to investigate the appropriateness of the default vehicle speed distribution vehicle. The following are the VISSIM default speed distributions which gave the initial percentage following and passing rates. The red line (with triangle markers) is for cars while the purple line (with rectangle markers) is for trucks.

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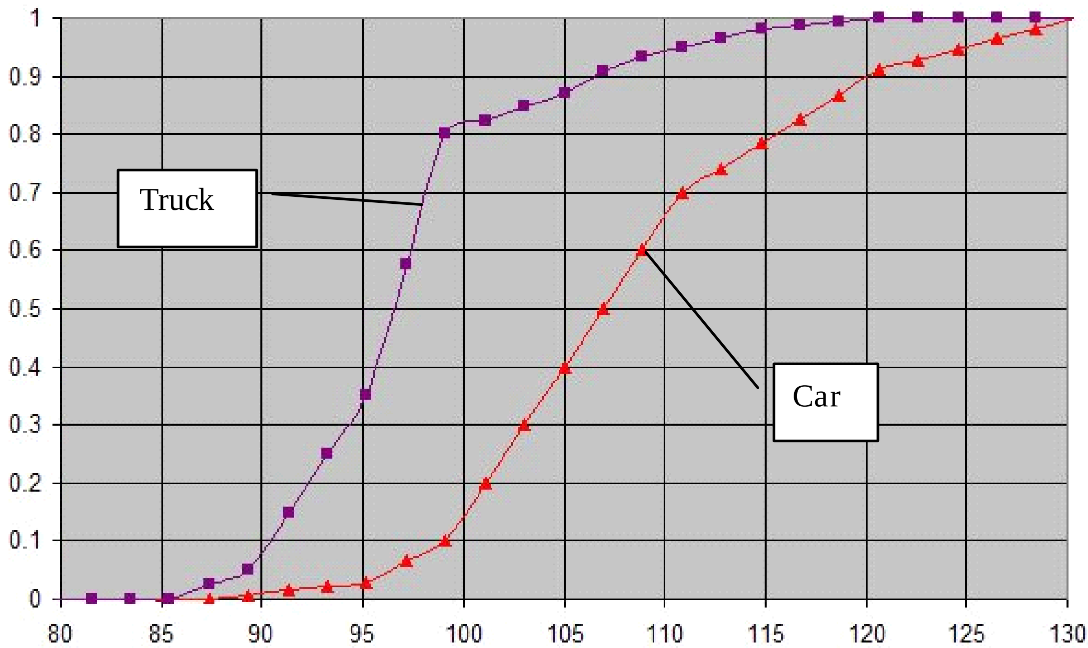
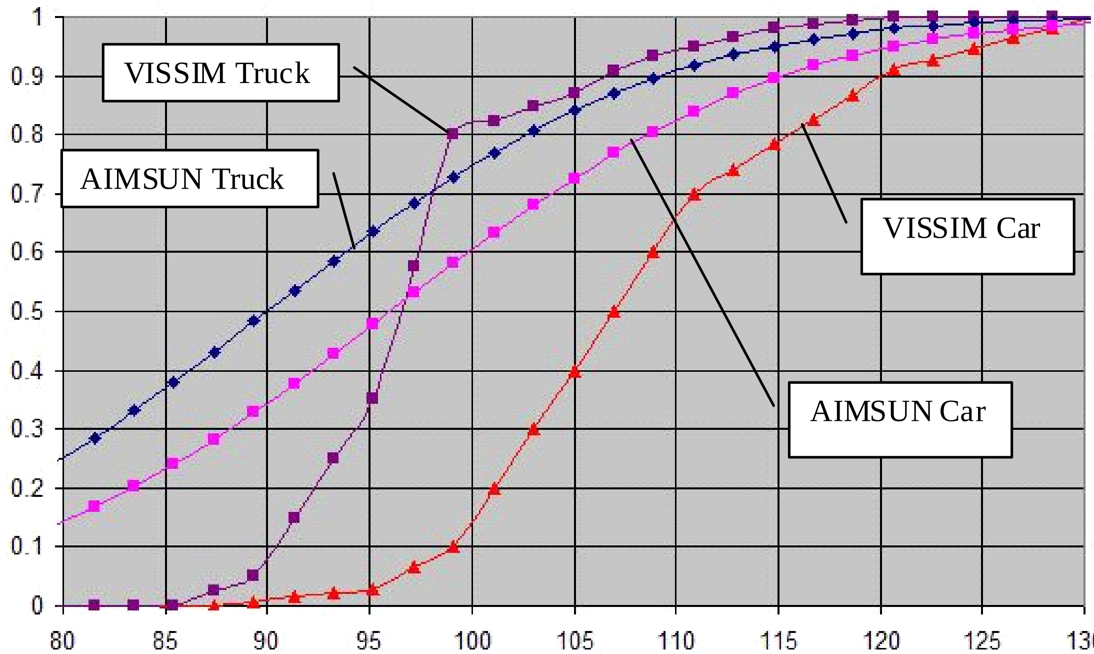
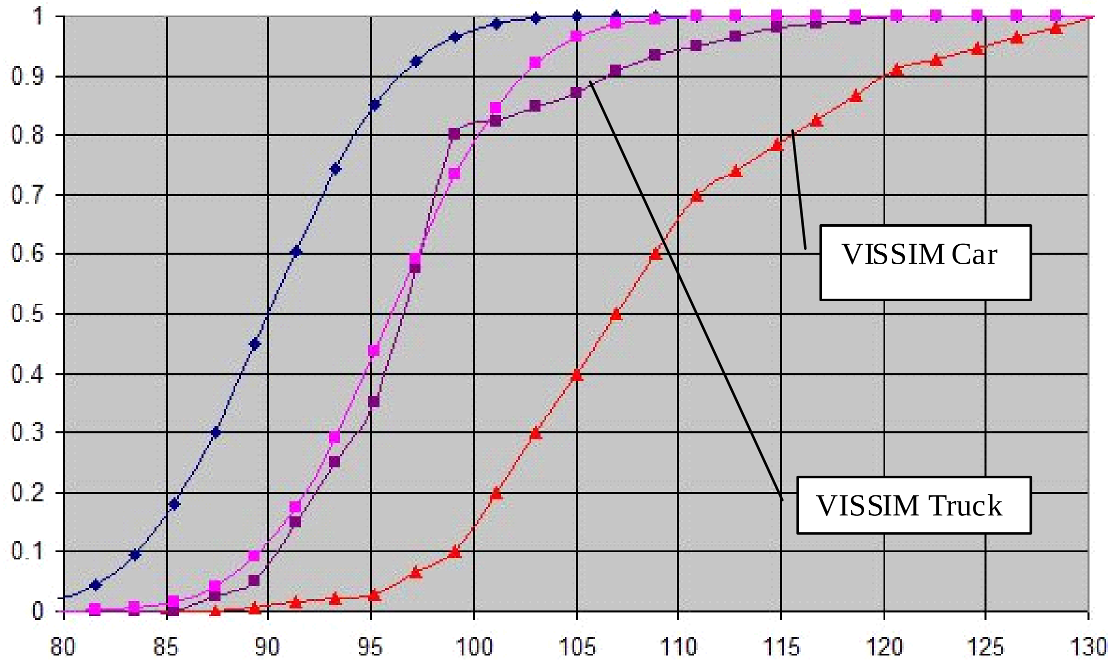
[2] The passing rate at Data Collection Point #4 was plotted to a polynomial – $y = 0.00035x^2 + 0.251x$ – in the literature. Where *y* is the number of passes and *x* is the one-way hourly volume.



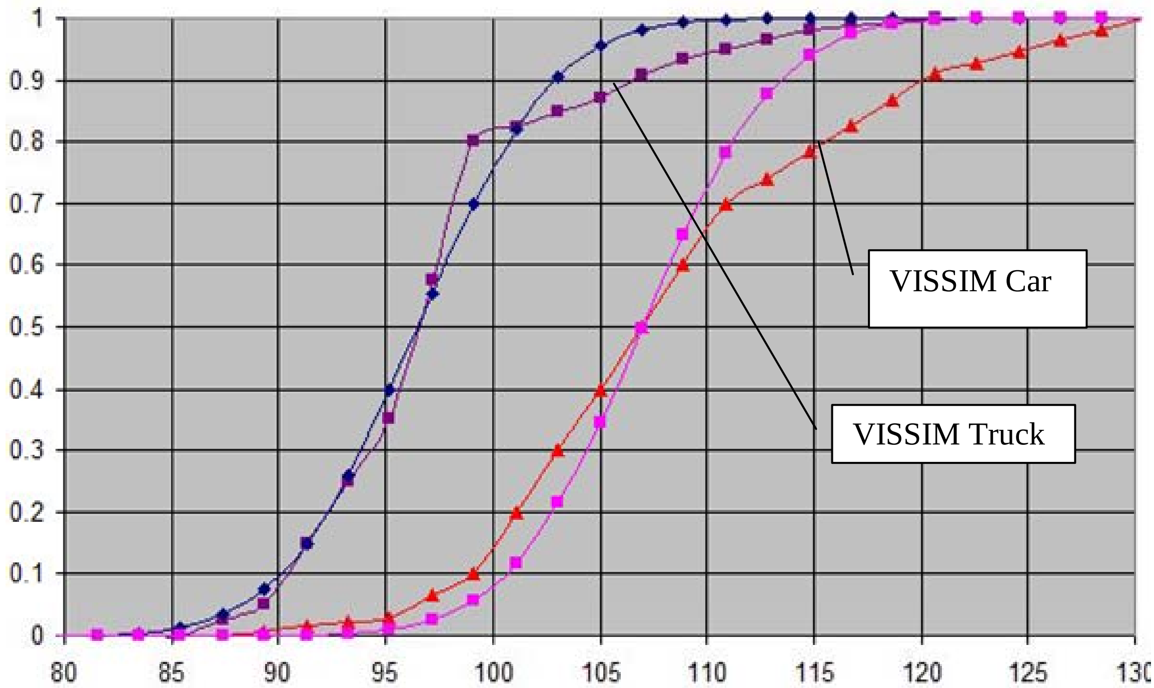
Firstly, the speed distributions from an AIMSUN model, which were calibrated to an urban 100km/h environment were tested. These are normally distributed with means of 96 and 90 km/h for cars and trucks respectively and a standard deviation of 15 km/h applied for both. The following graph illustrates the differences between the VISSIM default and the AIMSUN distributions:



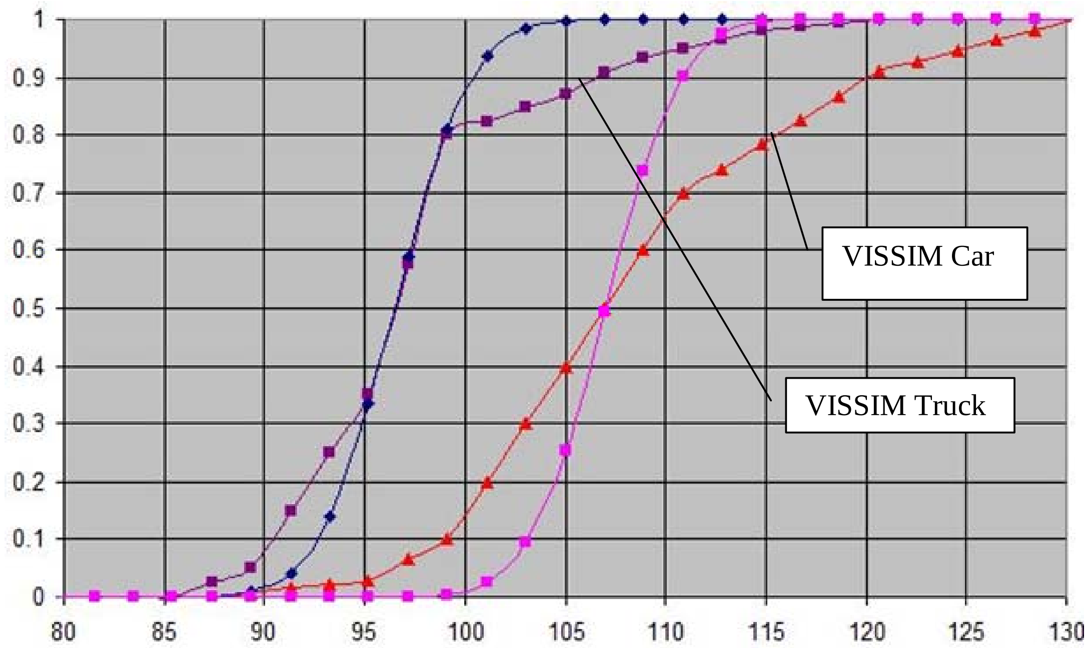
The model outputs with the AIMSUN speed distributions showed an upward shift of headway distribution and an increase of percentage following of 10-15%. A possible explanation could be the wider distribution as compared with that of the VISSIM default distribution, resulting in higher speed differentials between vehicles. From the above, the speed distributions with the same mean value as the AIMSUN speed distributions but with only a 5 km/h standard deviation were tested.



The above speed distributions showed similar headway distribution and % following as the VISSIM default. Another speed distribution was also tested using the mean of the VISSIM default but with 5 km/h standard deviation.



These results showed a better fit of headway distribution and percentage following. This can be explained by the new speed distributions having about the same proportion of vehicles travelling below the mean values while less vehicles travelling faster than the mean speed, i.e. less speed differentials between vehicles. The standard deviation of the speed distributions were then changed to 3 km/h.



This further improved the goodness of fit to the benchmark data. However, visual checks on the model found that due to the smaller speed differential, many vehicles were not able to complete their overtaking manoeuvres. This is also due to the limitation in VISSIM where the user has little control over vehicle overtaking behaviour. The "following" vehicles in VISSIM will always want to overtake the preceding vehicles as long as the speeds of the "following" vehicles are faster than the preceding vehicles.

It was then decided that since speed distribution alone would not give a better calibration result while showing realistic behaviour, the speed distribution should be fixed at a value that could be justified from research data. After consulting the literature, it was decided to use the mean values as published in Bennett (1985)[3] of 97.3 km/h and 84 km/h for cars and trucks respectively, with a standard deviation of 12.75 km/h; instead of using normal distribution it was decided to use a 5 km/h standard deviation for the lower part (<0.7) of the speed distribution. This made the speed distribution graph similar to the VISSIM default. The following two graphs show the resulting speed distributions for cars and trucks.

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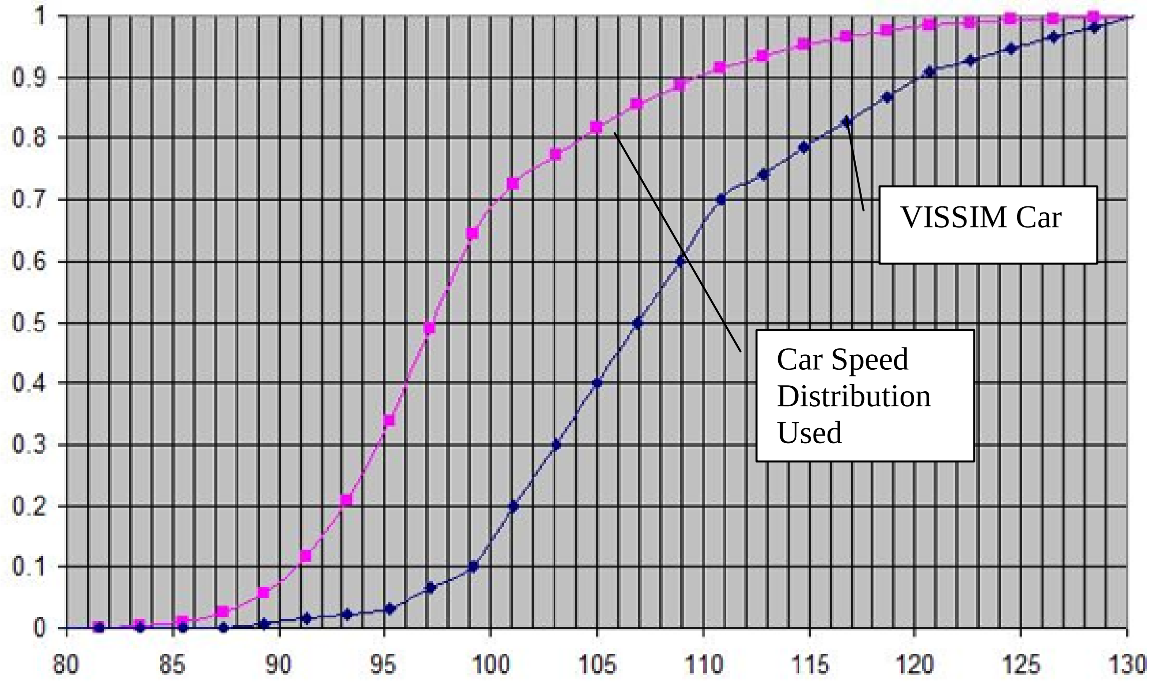
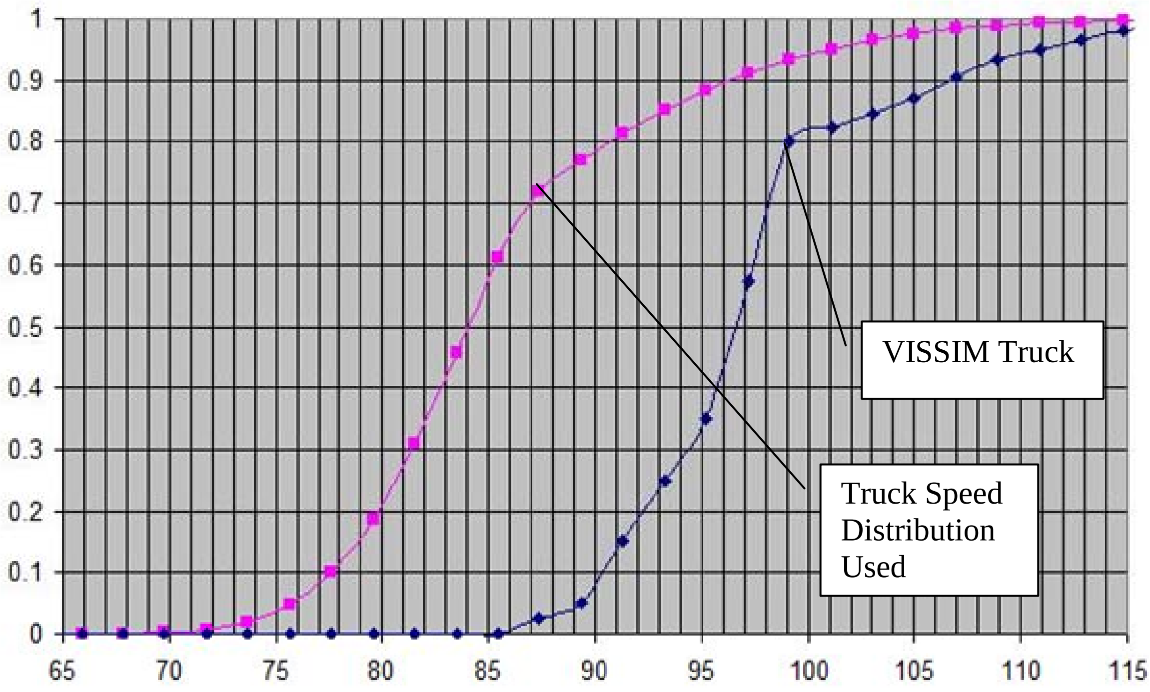
[3] Bennett, C.R. (1985). *Vehicle Speeds on Rural Highways in New Zealand*. National Research Board, RRS-007.





A2.3.3 Effect of Percentage HCV

The HCV proportion was increased to 10% to obtain an initial indication of the impact of varying this parameter. The results showed a slight increase in percentage following, but no perceivable differences in the headway distribution.

A2.3.4 Positioning of Data Collection Points

The data collection points #1 and #5 were moved upstream to 200 metres from the start of the passing lane and downstream 200m to the end of the passing lane to reflect the position as in

Cenek and Lester (2008). The results showed no difference at all and hence it was concluded that the position of these two data collection points at their original positions (at 100m) will not affect the results.

A2.3.5 Car-Following Model Parameters

Two parameters within the VISSIM car-following model were tested: (i) CC1, which is the desired time headway (in seconds) to the preceding vehicle; and (ii) CC2, which is the following variation in metres. It is important to note that CC1 and CC0 (the distance between vehicles when the speed is zero; default value is 1.5m) define the safety distance which is the minimum distance a driver will keep while following another vehicle.

CC1 was first increased from the default value (0.9 seconds) to 1.95 seconds to reflect the recommended safety distance of 2 second by the New Zealand recommended driving practice. The entire headway graph shifted to the right by 1 second and also upwards; the shift of upward direction seems to be directly proportional to flow. The percent following also increased at higher flow. The increase in percent following can be explained by the increase in the length of platoon as each vehicle is keeping a greater distance from the preceding vehicle, increasing the platoon length, so increasing the likelihood of vehicles being platooned at a particular flow volume.

The default CC2 value is 4 metres which represents a headway variation of about 0.15 sec at 100 km/h. It was increased to 27.8 metres to reflect a 1 second headway variation. The results showed negligible effect on both the headway distribution and percentage following. As this was shown not to be a sensitive parameter in the context of passing lanes, it was decided to keep CC2 at its default value

As Lay (1986) showed an average headway of about 1.5 seconds as a best estimate, it was decided that a CC1 value of 0.9 seconds was too small (this represents a safety distance of 0.95 seconds). Also the simulation did not provide a distribution of headways that reproduced those from the field research for platooned flow (looking at headway distribution in categories of < 1 second, 1-2 seconds, 2-3 seconds, 3-4 seconds). The CC1 value was increased to 1.45 seconds and CC2 to 10 metres which gave a good fit to the field observations.

A2.3.6 Reaction Time

The simulation resolution in VISSIM implies the reaction time of the drivers, for example a simulation resolution of 1 time frame per simulation second implies a 1 second reaction time (1/1) while a simulation resolution 2 implies a 0.5 second reaction time (1/2) etc. Research has shown reaction times between 1 second when a drivers is fully alerted (i.e. expecting the event) up to 2.5 seconds for an unaware driver. The simulation resolution in this case assumes attentive aware driving and 1 time frame per simulation second.

A2.3.7 Lane-Changing Parameters

With the car-following parameters and speed distributions broadly determined, the capacity of the unassisted passing lane is then calibrated by testing the lane changing model parameters in VISSIM. The merging behaviour of vehicles using the VISSIM default values showed vehicles which could not find sufficient gap after overtaking would come to a standstill at the end of the merge, particularly at high flow rates. The programme then removes these vehicles from the system after a certain time. This is clearly an undesirable modelling response.

The following lane changing parameters were determined to be the critical ones, and were modified to replicate the merging behaviour at passing lanes more realistically.

Lane-Changing Behaviour

Parameter	Description	Default Value	Changed Value
Waiting time before diffuse	Defines the maximum time that vehicles will wait at the emergency stop position while looking for a gap to merge before being removed from the network	60s	9999s (i.e. vehicles are not removed from the network)
Minimum Headway	Defines the minimum distance gap to the vehicle in front for a lane change in a standstill condition	0.5m	0m (increases the aggressiveness of forced merging)
Safety distance reduction factor	This reduces the safety distance of vehicles while lane changing occurs	0.6	0.6
Maximum deceleration for cooperative braking	This defines the maximum deceleration of the vehicles while allowing a lane changing vehicle to change into its own lane	-3 m/s^2	-6 m/s^2 (allows vehicles travelling in the main lane to decelerate faster, in order to let in a merging vehicle).

The safety distance factor was not changed after testing different values. It is listed here because it essentially defines the gap acceptance in conjunction to headway time in the car following model. That is, to change gap acceptance, one could change either the headway time in the car following model or change this parameter in lane changing model or both.

There are other parameters in the lane-changing model which were tested but insensitive to passing lane situations. The result of calibrating the lane-changing parameters was that the merging capacity (i.e. where the permanent tailback effect starts occurring) is at about 1300 veh/h, which is consistent with the literature review.

A3 Base Model Development

A base model was developed for each of the four passing lane configurations. They shared the same set-up parameters of (i) 5 km single lane section before the passing lane; (ii) diverge taper of 100 metres; (iii) passing lane of 1250 metres; (iv) merge taper of 160 metres; (vi) 5 km single lane section after the passing lane; (vii) 10% HCV and all HCVs are restricted to the slow lane only except in the merging section. Characteristics of the four layout options for comparing ITS-assisted merges were:

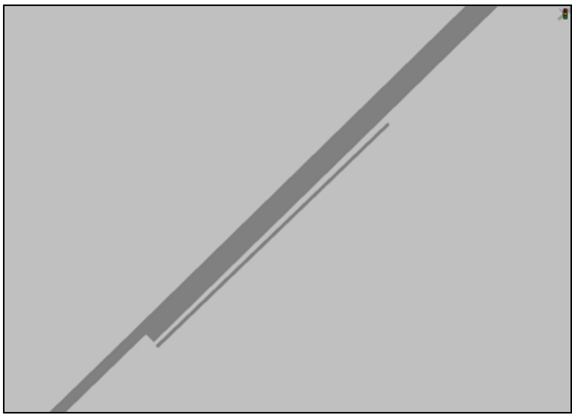
Unassisted Merge - for the unassisted passing lane, the "lane end" decision point was set at 200 metres before the merge taper start. This means that the vehicles will start signalling and look for gaps in the main lane from this point onwards.

Closed Passing Lane - all vehicles were simply banned from using the overtaking lane within the two-lane passing section.

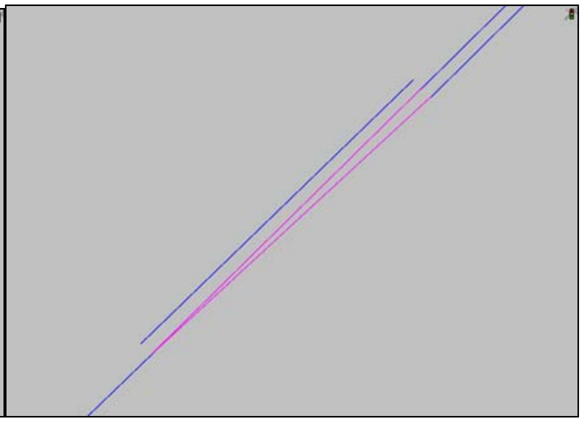
ITS assisted Early Merge - the effect of VMS signage is modelled using the compulsory lane change parameter at a distance from the merge taper, set at 660m to replicate the location of a VMS sign at 500 metres before the start of merge taper which says "overtaking traffic merge left/right". Otherwise the layout is the same as the unassisted merge.

ITS assisted Late Merge - the diverge and merge tapers were changed from the "lane start/end" arrangement to a "two connector" arrangement as illustrated below. Approaching vehicles were assigned to each lane equally with a 50/50 split (including HCVs being assigned on the left lane only).

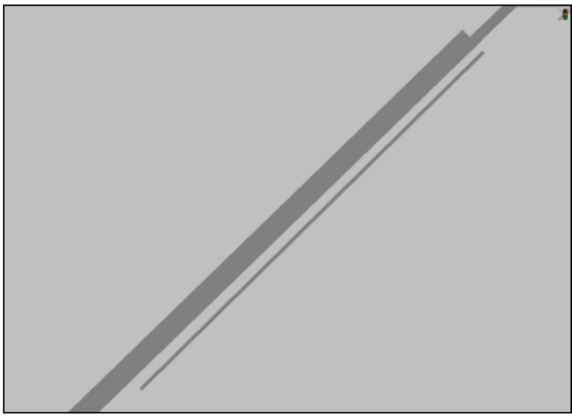
Lane Start Diverge



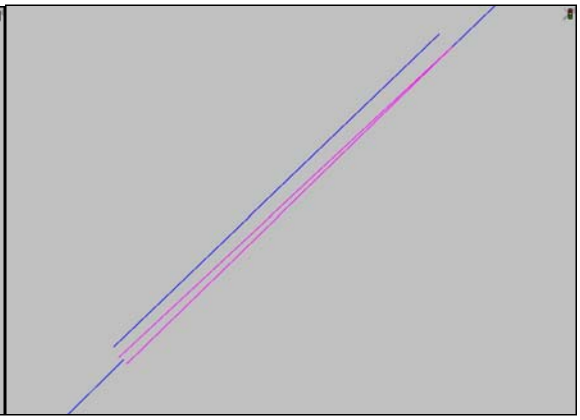
2 Connectors Diverge



Lane End Merge



2 Connectors Merge



The passing lane section was changed from a single two-lane section to two single-lane sections at the diverge point to model the effect of the VMS message “Traffic Congestion Use Both Lanes”. Similarly, the merge point was changed to model the VMS message of “Merge Here, Take Your Turn”. The traffic was channelled into two parallel but separated lanes.