

Assessing Passing Opportunities – Stage 3

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Assessing Passing Opportunities – Stage 3

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Executive Summary

Introduction

This research continues recent work to investigate ways of analysing and providing for improved passing opportunities on rural highways in New Zealand. It was carried out during 1999-2001. The main objectives of this research were:

- To assess the effectiveness of no-overtaking delineation using modified (horizontal curve) warrant criteria.
- To improve the Transfund New Zealand simplified procedures for passing lane analysis using field surveys and TRARR¹ modelling.
- To investigate the use and performance of slow vehicle bays using field surveys.
- To develop a framework for future development of detailed rural simulation modelling in New Zealand.

Driver reaction to changed no-overtaking markings

Field surveys and subsequent desktop analysis of three short overtaking sites in New Zealand, together with literature review, revealed:

- Small amounts of overtaking (averaging up to 2.5% of all traffic) do occur at sites with sub-standard horizontal sight distance (SD). The maximum potential amount of overtaking occurring is inversely proportional to the opposing traffic.
- Changing no-overtaking markings to include horizontal curve criteria would at least double the amount of marking required. It would also reduce overtaking rates by up to 50%, and increase time spent following by 20-30%. However travel times would increase by no more than 9%, and less where traffic is greater. Likely changes in driver compliance do not have a significant effect on the predicted changes.
- The predicted reduction in “dangerous” overtaking manoeuvres will be somewhat offset by the increase in driver frustration. However a net safety benefit is expected with the introduction of more restrictive no-overtaking criteria.
- Allowing no-overtaking lines to be marked on straight approaches to isolated horizontal curves, to inform drivers when suitable overtaking lengths are no longer available, may provide a suitable compromise between the existing vertical-only SD criteria and a full combined horizontal/vertical SD criteria.

Field calibration of simplified procedures

Field surveys, literature review, and various TRARR and simplified procedure analyses on passing lane situations showed:

- The recent updating of US Highway Capacity Manual (HCM) procedures provide a fairly straightforward means of assessing the effects of a passing lane on vehicle bunching and travel speeds. It may be reasonable to use them to assess passing lanes in New Zealand, either at a project-specific level or for strategic route studies.
- A revised Willingness-To-Pay (WTP) relationship has been derived that relates driver frustration to changes in Percent Time Spent Following (PTSF).
- A number of improvements to the existing simplified passing lane procedure were identified. These include reducing calculated passing demand as the Accrued Passing Demand (APD) increases, and revising maximum passing rates.

¹ TRARR - TRAFFIC ON RURAL ROADS ARRB (AUSTRALIAN ROAD RESEARCH BOARD) COMPUTER MODEL PACKAGE FOR RURAL ROAD SIMULATION

- Even with the suggested changes, the simplified procedures still tend to over-estimate increases and decreases in APD. What appears to be missing is some “equilibrium” value that a given road and traffic situation will gravitate towards.
- The relationship between APD and bunching is still not entirely certain, and maybe the simplified procedure should be re-presented in terms of %Bunching instead.

Field performance of slow vehicle bays

Field surveys and subsequent desktop analysis of eight slow vehicle bay (SVB) sites in New Zealand, together with a literature review, revealed:

- SVB use appears to be very dependent on the location and design of each site. The use of SVBs by platoon leaders generally ranges between 30-60%, a higher rate than that found in overseas studies.
- SVBs do not greatly reduce the proportion of bunched vehicles, particularly in winding alignments, with less than 10% (absolute) reductions observed. The short-term benefits however probably do provide some reduction in driver frustration.
- Trucks and recreational vehicles typically made up 70-90% of all vehicles using SVBs, with trucks in particular being high users. Some sites that looked more like passing lanes had higher car use.
- Evidently confusion by drivers exists over the use of some SVBs, as seen in the numbers of lone vehicles using them and in conflicts at SVB merges.
- The Borel-Tanner Distribution provided an excellent model of bunch sizes observed at SVBs. Using this model, queues of two or more vehicles-following play a significant part when considering the operation of SVBs.
- The current New Zealand guidelines for SVB lengths may be inappropriate, given the number of merge area conflicts and the likely number of multi-vehicle queues. For a short SVB with sufficient queuing, any travel time benefits gained by the passing vehicles may be negated by the delay placed on the overtaken vehicle.
- SVBs modelled by TRARR provide a realistic reduction in the proportion of platooned vehicles, but under-estimate the actual travel-time savings.
- A simplified formula has been developed that appears adequate for predicting bunching rates after an SVB, given initial on-site field surveys, and can be applied to the calculation of the frustration benefits from a proposed SVB.
- The safety benefits of SVBs are not entirely clear. Downstream crash savings beyond the site may be very limited, but savings are likely at the site if the new SVB provides a safer alternative to previous overtaking attempts at that site.

Development of framework for detailed modelling

Literature review and assessment of various highway simulation models identified a number of observations:

- There is a need to consider in a unified manner the overall impacts of changes to road alignments and cross-sections, for both safety and efficiency.
- Simulation would still appear to be necessary when evaluating realignments, multiple passing lanes, continuous four-laning, and when validating “simplified” methods.
- TRARR has some conceptual problems when evaluating some local scenarios. Also more practical concerns are with the TRARR software and documentation.
- A US software tool, TWOPAS, holds some promise as a possible replacement for TRARR. However, work is needed to confirm its validity and practicality for its use in New Zealand.

An alternative approach is to consider the feasibility of applying more recent micro-simulation tools that are being used for urban networks and inter-urban motorways overseas, to rural highways in New Zealand.

- A general framework has been developed in our research that provides an overview of what an ideal rural simulation model needs to take account of, in terms of how various road, traffic, vehicle and driver inputs interact to provide all the information required for project evaluation in New Zealand.

Recommendations

Key recommendations are:

Driver reaction to changed no-overtaking markings

- Further overtaking surveys or driving simulator studies should be used to assess revised no-overtaking marking criteria.
- The use of a speed-dependent measure for assessing no-overtaking areas should be considered in New Zealand.
- A modified criterion allowing no-overtaking lines to be marked on straight approaches to isolated horizontal curves should be investigated.

Field calibration of simplified procedures

- The revised US HCM procedures for passing lanes should be investigated further in New Zealand to confirm their appropriateness for evaluating passing lane projects.
- The revised driver frustration WTP procedure and suggested changes to Transfund's simplified passing lane procedures should be adopted.

Field performance of slow vehicle bays

- SVBs should be clearly marked distinctly from passing lanes, to prevent confusion by drivers. Driver education about SVBs should also be carried out.
- SVBs should be located only where the mean traffic speed is less than about 60km/hr, and even lower speeds where greater traffic volumes and longer queues are likely.
- SVB travel-time savings should only be considered where the site does not cause undue delay to those vehicles being overtaken that are waiting to re-enter the traffic stream.
- An alternative simplified evaluation procedure should be used for SVBs based on these research findings, instead of the existing simplified passing lane procedures.

Development of framework for detailed modelling

- The most recent versions of the TRARR and TWOPAS models should be compared in a range of New Zealand settings and scenarios with actual field data.
- Further work should incorporate safety assessments in rural simulation models used in New Zealand, including an investigation of the US IHSDM² project.
- An improved rural simulation model should either be adopted or developed, following the further investigation of the various alternatives.

² IHSDM – Interactive Highway Safety Design Model

Abstract

This research continues recent work to investigate ways of analysing and providing for improved passing opportunities on state highways in New Zealand. It was carried out during 2000-2001 and involved field surveys, literature review, TRARR modelling and subsequent desktop analysis. The effectiveness of no-overtaking delineation using a modified (horizontal curve) warrant criterion was assessed. Changes to Transfund New Zealand simplified procedures for passing-lane analysis were suggested to improve the robustness of the model. The use and performance of slow vehicle bays were investigated and alternative evaluation procedures developed. Various highway simulation models were assessed and a framework devised for future development of detailed rural simulation modelling on New Zealand rural roads.

1. Introduction

This research continues recent work carried out to investigate ways of analysing and providing for improved passing opportunities on rural highways in New Zealand. In doing so, it must balance efficiency and safety demands. The project was carried out during 1999-2001. The specific objectives for this stage of the work are outlined in Section 1.3 of this report.

1.1 Background

New Zealand's relatively rugged terrain and low traffic volumes have meant that virtually all its rural strategic routes have been built as two-lane highways. As traffic volumes have increased, increasing pressures have been placed on maintaining an adequate level of service. Passing opportunities, such as passing lanes and slow vehicle bays (SVBs), provide a means to relieve these pressures and their construction is greatly encouraged by the general public.

At the same time, two-lane road sections unsuitable for passing (because of their poor alignment) must be identified and marked accordingly to assist drivers in safe passing manoeuvres. No-overtaking lines have become widely accepted and important traffic control devices in many countries, including New Zealand.

Passing opportunities generate economic benefits by reducing travel times, as they release impeded vehicles from platoons¹. Released drivers may then travel at their desired speed until they once again become trapped in slower moving platoons. The length of road over which passing opportunities are effective is, therefore, generally much greater than the physical length of the passing opportunity section. When drivers are unable to overtake a slower vehicle they are likely to become frustrated. This can lead to an increase in unsafe passing manoeuvres that can lead to crashes. Thus provision of passing opportunities allows for safe passing manoeuvres and a subsequent reduction in crashes.

Transfund New Zealand (Transfund) currently requires that all new roading projects be assessed using their standard economic evaluation procedures to produce a "Benefit-Cost Ratio" (BCR). BCRs assess the tangible benefits related to savings in travel-time, vehicle-operating costs, and crashes, against the construction and maintenance costs involved (Transfund 1999).

Some overseas national or state jurisdictions justify passing lane construction by means of warrant requirements, examples of which can be seen in AUSTRROADS (1993). However Transfund's requirements mean that specific benefits must be calculated, usually by means of a rural road simulation model, such as ARRB Transport Research's TRARR² package (Shepherd 1994).

¹ Platoon – moving group of queued vehicles led by a slower vehicle, also known as a "bunch".

² TRARR – TRAffic on Rural Roads ARRB (Australian Road Research Board) computer model package for rural road simulation.

Nevertheless, as a modelling tool for passing opportunities, TRARR has proved to be an adequate package. In particular, the ability to import RGDAS³ (road geometry) data from New Zealand state highway network has been particularly effective. However a number of potential drawbacks have been identified, limiting TRARR's use for all rural simulation work in New Zealand. It is a relatively specialised tool and, despite recent improvements in its data collection requirements, it is still fairly time consuming (and hence costly) to make use of. There is therefore scope to provide an improved rural simulation tool for use in New Zealand.

1.2 Previous New Zealand Research

Initial research in New Zealand (Tate 1995, Thrush 1996) investigated ways in which improved passing opportunities may be provided and analysed, and included a literature review of the methods available to assess the benefits of improved passing opportunities. The study, based on a series of desktop analyses, concluded that:

- The analysis of the benefits of improved passing opportunities is a complex task that requires consideration of a number of variables.
- Although simplified graphs of likely benefits were produced, the simplifications involved reduced the usefulness of these graphs.
- The use of simulation models such as TRARR was still the most appropriate tool in New Zealand for detailed passing lane analysis.
- Applying the simulation models with differing degrees of refinement can provide a staged assessment process. However, a more simplified means for at least preliminary assessment was still desired.
- The safety implications of improved passing opportunities were unclear. An investigation into the potential safety implications should be undertaken.

Stage 2 of the research project (Koorey et al. 1998) built on the initial work, and investigated the development of a simpler system to determine the need for, location of, and benefits to be derived from passing lanes. Other aspects of the research also looked at measures of driver frustration resulting from inadequate passing opportunities, the crash reduction potential of passing lane improvements, and optimum data requirements to calibrate TRARR. The key findings of the study were:

- A tangible Willingness To Pay (WTP) for passing opportunities related to driver frustration was determined. WTP was found to be significantly correlated with the easily measurable "Unsatisfied Passing Demand", although further work was required to refine the relationship.
- A number of factors were found to have an effect on passing lane crash rates, including crash type, crash location, and passing lane length. Typical mid-block injury crash rates for three/four-lane rural highway sections in New Zealand were found to be on average 25% lower than the equivalent two-lane rates.

³ RGDAS – Road Geometry Data Acquisition System: an ARRB proprietary text format.

- A simplified model for assessing the optimum location of passing lanes was developed. The model is based on comparing the supply of and demand for passing opportunities along a route and requires less input data and analysis time than TRARR.
- A comparison of both the simplified model and TRARR with “before and after” field data was made. Both analyses were found not to over-estimate actual travel time benefits derived from passing lanes.

The driver frustration WTP and methodology for the simplified model were incorporated in Transfund’s 1999 *Project Evaluation Manual* (PEM). However, a number of problems requiring further work were identified. These included evaluation of other means of providing passing opportunities (such as SVBs), further development of a variable WTP, and confirmation (using field surveys) of theoretical values used in the simplified procedures.

At the same time, Transit New Zealand commissioned separate research investigating the existing warrants⁴ for no-overtaking lines in New Zealand (Thrush et al. 1998). Currently no-overtaking delineation in New Zealand is generally only used on sections of road where sight distance is restricted by vertical curvature. Concern has been expressed regarding the adequacy of current no-overtaking line criteria in New Zealand, particularly with regard to horizontal curves and higher speeds.

Thrush et al. recommended that a review of the sight distances used for marking no-overtaking lines be undertaken. In particular they suggested that a sight distance approach based on the “critical position”, a point at which the decision to complete the manoeuvre affords the same clearance as the decision to abort. They also recommended that no-overtaking warrants in New Zealand should take into account the actual operating speeds on the highway. Further work was required however to assess the impacts of changed criteria on road-marking costs and driver efficiency and safety.

1.3 Objectives

The main objectives of this research were:

- To assess the effectiveness of no-overtaking delineation using modified (horizontal curve) warrant criteria.
- To improve the Transfund simplified procedures for passing lane analysis using field surveys and TRARR modelling.
- To investigate the use and performance of slow vehicle bays using field surveys.
- To develop a framework for future development of detailed rural simulation modelling in New Zealand.

⁴ Warrant – a series of standards or minimum levels that must be met before justifying a traffic engineering requirement.

1.3.1 Task 1: Driver Reaction to Changed No-overtaking Markings

A quantitative field survey was carried out to determine driver overtaking behaviour on short lengths of visible straight highways. The sites chosen display some level of overtaking despite a moderately curved alignment. A second stage to re-mark the sites for no-overtaking using the horizontal curve warrant criteria is discussed.

Additionally a desktop analysis was carried out on the effect of changed no-overtaking criteria on marking requirements, and safety and efficiency. The behavioural implications of these findings, in conjunction with previous driver overtaking research, are also assessed.

1.3.2 Task 2: Field Calibration of Simplified Procedures

The existing simplified passing lane procedure was reviewed, in conjunction with field data. In particular, two key components were examined:

- How the initial and subsequently accrued passing demand relates to corresponding bunching levels.
- The maximum possible overtaking rate available within passing lanes.

The field data were compared against the expected results calculated using the PEM simplified procedures, and TRARR simulation models. In addition, a review of the driver frustration Willingness-To-Pay (WTP) was made by approaching the original survey data from the perspective of Percent Time Spent Following (PTSF). Suggested modifications to the simplified procedure methodology are made.

1.3.3 Task 3: Field Performance of Slow Vehicle Bays

The typical use of slow vehicle bays either way to reduce passing demand has been established using field surveys. The implications with respect to SVB lengths and their safety performance are also investigated.

Analysis of the data attempted to relate the field performance to TRARR models and the previously prescribed simplified procedures for passing lane analysis. Suggested modifications for their application to SVBs are made.

1.3.4 Task 4: Development of Framework for Detailed Modelling

The existing features and deficiencies of TRARR, and of other common simulation tools world-wide, are summarised and compared. This aspect of the research is based on literature review, correspondence with relevant organisations, and the personal experience of the researchers.

From this, a framework of ideal features for detailed modelling of New Zealand rural roads is established. The resulting prescription is envisaged to form the basis of a specification brief, should future development of such a modelling tool be undertaken.

1.4 Report Outline

Section 2 of this report examines the effects of no-overtaking delineation using modified (horizontal curve) warrant criteria.

Section 3 then details findings from field surveys and data analysis of passing lanes and suggests improvements to the Transfund simplified procedures for passing lane analysis.

Section 4 reports on field surveys looking at the use and performance of slow vehicle bays, and suggests changes to project evaluation procedures for them.

Section 5 discusses detailed rural simulation modelling in New Zealand and develops a framework for future development.

Conclusions and recommendations from this report are summarised in Sections 6 and 7, followed by the References, and Appendices.

2. Driver Reaction to Changed No-overtaking Markings

2.1 Introduction

No-overtaking lines have become widely accepted and important traffic control devices in many countries. Their primary function is to prohibit overtaking manoeuvres where crossing into an opposing traffic lane is almost always above an acceptable risk threshold.

Over the years, research into warrants for no-overtaking lines has produced alternative criteria for line marking and questioned the basis for warrants that were developed some decades ago. Concern has been expressed regarding the adequacy of current no-overtaking line criteria in New Zealand. Specific concerns relate to the lack of marking for horizontal visibility criteria and the increase in speeds following the raising of the open road speed limit in 1985 (Goh & Barnes 1988).

The three basic types of visibility restriction on two-lane rural roads are those related to vertical curvature, horizontal curvature, or a combination of both horizontal and vertical curvature. However, in New Zealand, none of the no-overtaking lines are marked solely for restrictions in horizontal alignment (TNZ/LTSA 1997). Although some sites are marked because of an “unusual combination of vertical and horizontal curves”, this is a subjective criteria only. The main argument supporting the New Zealand system is that drivers are able to see that a horizontal curve will conceal oncoming traffic, whereas a driver cannot tell if a vertical curve will conceal oncoming traffic or not.

Research suggests however that, although a driver will be able to see that a horizontal curve will hide oncoming traffic, they will probably under-estimate the time and space required for overtaking (Gordon & Mast 1968). Also, as the driver’s speed increases, the difference between their estimated distance and the actual distance required increases. Although no-overtaking lines can provide this information, the question is whether drivers would accept and abide by additional markings.

Another study showed that, although drivers were able to take their own speed into account when deciding whether or not to overtake, they could not estimate oncoming vehicle speed (Faber et al. 1968). They too suggested a need for the sight distance (SD) criteria to become more conservative with increased speed, as the research showed that driver judgement worsens with increased speed.

These two studies strongly prove that no-overtaking lines are a necessary safety device on the highway. The evidence also points to the need to relate overtaking sight distance to vehicle speed.

According to the data obtained from overtaking injury crashes on New Zealand rural two-lane roads between 1996 and 2000, only 5% of overtaking crashes were

recorded at locations where visibility was restricted but had no marked no-overtaking lines. These crashes however accounted for 14% of the annual costs of overtaking crashes, underlining the seriousness of not judging overtaking distances correctly.

Another 15% of overtaking crashes occurred at locations with no-overtaking markings, accounting for 22% of the annual costs. This highlights the fact that non-compliance of no-overtaking markings will always be an issue. Increasing the level of enforcement of the regulations relating to the markings may improve their effectiveness.

2.2 Methodology

A quantitative field survey was undertaken to relate driver behaviour with changed no-overtaking delineation. Existing sites without no-overtaking lines would be observed as initial surveys. Subject to the results from these initial surveys, the sites would then be re-marked using the horizontal curve warrant criteria, increasing the amount of no-overtaking length delineated.

Sites on rural state highways were targeted for investigation. Ideally, the sites chosen should display a significant level of overtaking despite a moderately curved alignment. Suitable sites were selected using the following influencing variables:

- Proportion of local traffic (i.e. who are familiar with the road),
- Level of existing overtaking,
- Amount of adjacent passing opportunities,
- Traffic volumes (to obtain sufficient data),
- Accessibility to surveyors.

2.2.1 Site Selection

Preparing a shortlist of potential sites was the first challenge. The approach taken was to identify short lengths of state highways with relatively high crash numbers for overtaking-related crashes. This would indicate sites with possible problems due to relatively high rates of overtaking despite the poor alignment.

Initial inspection of the crash data revealed 41 sites with at least four overtaking-related injury crashes per kilometre since 1980. While it is accepted that many of the sites may have significantly changed over that period, this approach helped us to identify a sufficiently large initial sample to start from. Appendix A1 lists the original sites. Local Opus offices in the vicinity of each site were contacted for comments on the suitability of the selected sections, based on the criteria given in Section 2.1. This eliminated, for example, those sites that already had no-overtaking markings or other complicating features such as passing lanes or side roads. From this feedback, a shortlist of nine sites was identified for further investigation.

Road geometry data was used to calculate sight distances at sites, based on vertical sight distance alone (existing policy) and on both horizontal and vertical sight

distances (as the alternative policy). To analyse sight distances based on these criteria, the RGTRA software tool was used. RGTRA is an accessory program of ARRB Transport Research’s TRARR 4 package (for rural road simulation) that allows users to import road geometry data into TRARR. (Previously, creating TRARR’s ROAD files had to be done manually.) RGTRA converts “RGDAS” format *.XYZ files into *.TRA files that can in turn be imported into TRARR. The *.TRA files include, among other things, a calculation of forward and backward sight distances. As such, RGTRA is a useful tool in its own right for obtaining sight distance data.

The original 1992 road geometry data collected in New Zealand was stored in the appropriate “RGDAS” format. However the current road geometry data that is stored in RAMM⁵ is not in the correct RGDAS format, so a conversion routine is needed to produce equivalent *.XYZ files. Some code developed in an MS Access database enabled this. Both sets of data record, at 10m intervals (in both directions), the horizontal curvature, cross-fall (superelevation), and gradient.

As well as specifying a road geometry file to analyse, RGTRA also requires input information on the distances to the left and right offsets. These determine how much of an effect obstructions on either side are to the calculated SDs. For vertical-only SDs, a nominally large offset was used (1000m) reflecting the lack of effect of horizontal curve obstructions (i.e. similar to driving on an open plain). For combined SDs, an offset of only 5m was used, suggesting that the road is contained within a narrow “corridor” (i.e. similar to driving in a cutting).

The resulting sight distances were plotted together for comparison. Visual inspection of the plots for these allowed us to narrow down to three sites for survey. These sites were confirmed by final local inspection and are listed in Table 2.1.

Table 2.1 Locations of overtaking survey sites.

SH RS/RP*	Location	AADT	Description
SH2 RP99/15.2-116/0.0	North of Katikati, Bay of Plenty	8000	Tight curves but people pass when they can; 300m north of Stokes Road.
SH3 RP491/5.3-6.1	West of Woodville, Southern Hawkes Bay	6000	Straight just west of Old Cemetery Road, only passing opportunity between Woodville and Manawatu Gorge.
SH58 RP0/11.4-11.8	West of Pauatahanui, Wellington	6500	Short straight between tight curves. East of James Cook Drive.

SH – State Highway number; RS – Reference Station; RP – Route Position

AADT – Annual Average Daily Traffic volume

Abbreviated site names used in text are: (SH/RS) 2/116; 3/491; 58/0.

These sites do not appear to meet the current criteria for indicating with no-overtaking lines (and are not marked as such), but would be marked if a more restrictive horizontal + vertical criteria were used. Table 2.2 summarises the calculated sight distances.

⁵ RAMM – Road Asset Maintenance and Management system: road inventory database maintained by Transit New Zealand.

2. *Driver Reaction to Changed No-overtaking Markings*

Table 2.2 Sight distances (SD) (m) at the selected survey sites.

Site (SH/RS)	Increasing RP*				Decreasing RP*			
	Vertical SD Only		Horiz + Vert SD		Vertical SD Only		Horiz + Vert SD	
	Min SD	Ave SD	Min SD	Ave SD	Min SD	Ave SD	Min SD	Ave SD
2/116	199	786	159	230	150	632	159	225
3/491	449	932	130	305	429	931	130	307
58/0	219	790	79	175	200	734	79	207

* Increasing /Decreasing from RP, at start of highway

The average sight distance over the surrounding kilometre is seen to drop quite significantly when the combined horizontal/vertical criterion is applied, and falls below the current minimum requirement of 330m on rural roads. In most cases, the minimum sight distance at any point along the highway also drops significantly. Appendix A2 shows the sight distance plots for these three sites.

All three sites contain a relatively short straight, within tighter curves. Because of this, the sites also appear to be currently indeterminate for overtaking ability (i.e. some drivers will, some won't). The SH58 site contains a notable amount of "local" traffic (i.e. commuting between Hutt Valley and Kapiti/Porirua), and the SH2 and SH3 sites are less affected. Figure 2.1 shows one of the sites on SH58 (note that the lens foreshortens the length of highway seen).

Figure 2.1 SH58 overtaking survey site, near Pauatahanui, Wellington.

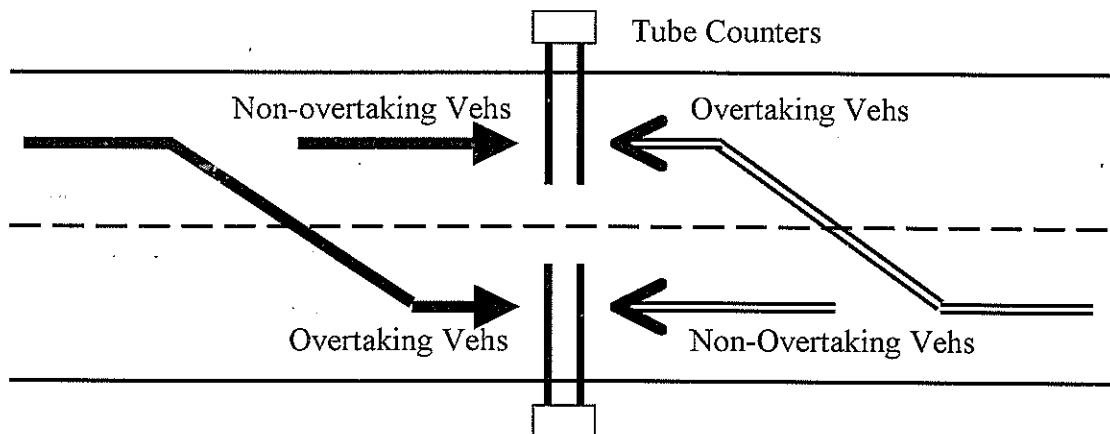


2.2.2 Site Surveys

At each of the three sites, automated traffic surveys were commissioned to record the occurrence of overtaking at these sites. This was done using pairs of tubes over each lane to identify the direction of each vehicle passing and hence split non-overtaking traffic and overtaking traffic. Traffic recorded travelling in the opposite direction to

the main flow of a lane was deemed to be overtaking in the opposite direction. Figure 2.2 demonstrates the surveying procedure.

Figure 2.2 Layout of overtaking survey sites.



The tubes were placed in the middle of the identified overtaking section, so as to pick up as many overtaking manoeuvres as possible. A typical 10-second manoeuvre at 108km/hr (30m/s) equates to 300m of length required, so it was expected that most overtakings in the vicinity should cross the tubes. It was possible that a vehicle could straddle the centreline and cross over both tubes, thus triggering a double count. However a gap was left between the opposing tubes to minimise this occurrence.

The surveying was carried out for at least a week at each site (the SH3 site was surveyed for two weeks). Unfortunately the SH2 site had road works approximately 300m south of the site, which may have influenced typical overtaking rates. The resulting traffic data was then summarised into quarter-hourly blocks to determine the level of overtaking occurring with a given traffic volume.

2.2.3 Analysis of Results

The hypothesis is that traffic flows, both in the same direction and in the opposing direction, have significant effects on the incidence of vehicles overtaking. A greater traffic flow in the same direction is more likely to result in having to follow slower vehicles and thus increasing the amount of overtaking desired. However a greater traffic flow in the opposing direction will limit the opportunities for safe overtaking and reduce the amount of overtaking achieved. The other factor that will influence overtaking rates is the proportion of road with adequate overtaking sight distance, including the amount of previous opportunities for overtaking.

Figures 2.3 to 2.8 plot the relationships between the overtaking rate in each direction and the traffic flow in the opposing direction for the three sites. The first thing to notice is the many instances when no overtaking occurs. And, indeed, the average overall level of overtaking at the surveyed sites seems quite low at 0%–2.5% of all vehicles. However this maybe a reflection more of the geometry of the sites chosen,

which provide only a limited (and arguably unsafe) overtaking opportunity. The fact that overtaking rates of up to 5% are occurring some of the time with opposing flows greater than 400 veh/hr (i.e. one vehicle on average every 9 seconds or less) is somewhat alarming. However vehicle platooning will also be occurring and this will often provide longer gaps in between bunches.

Figure 2.3 Overtaking rates (southbound) at Site 2/116, SH2 north of Katikati.

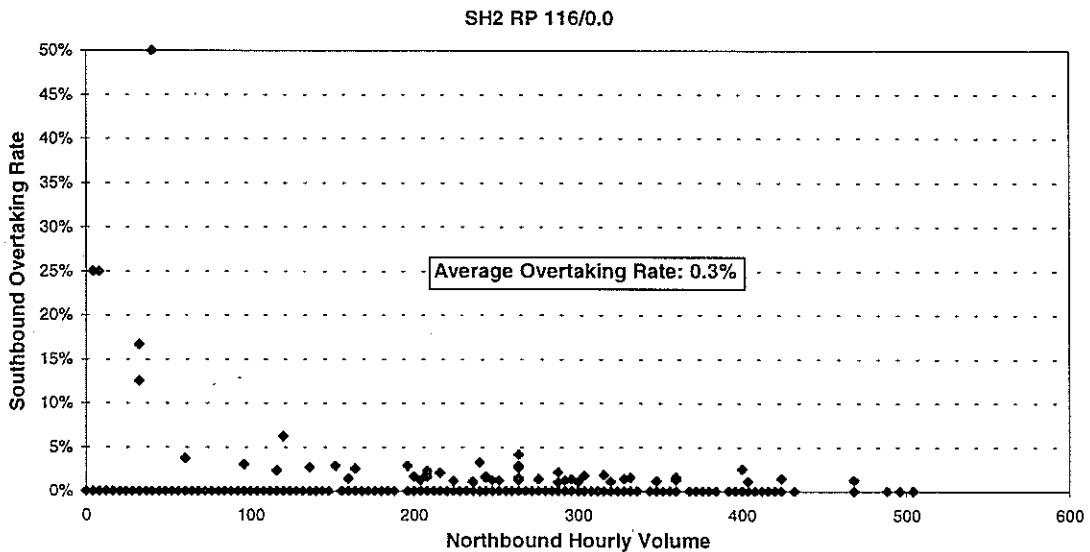
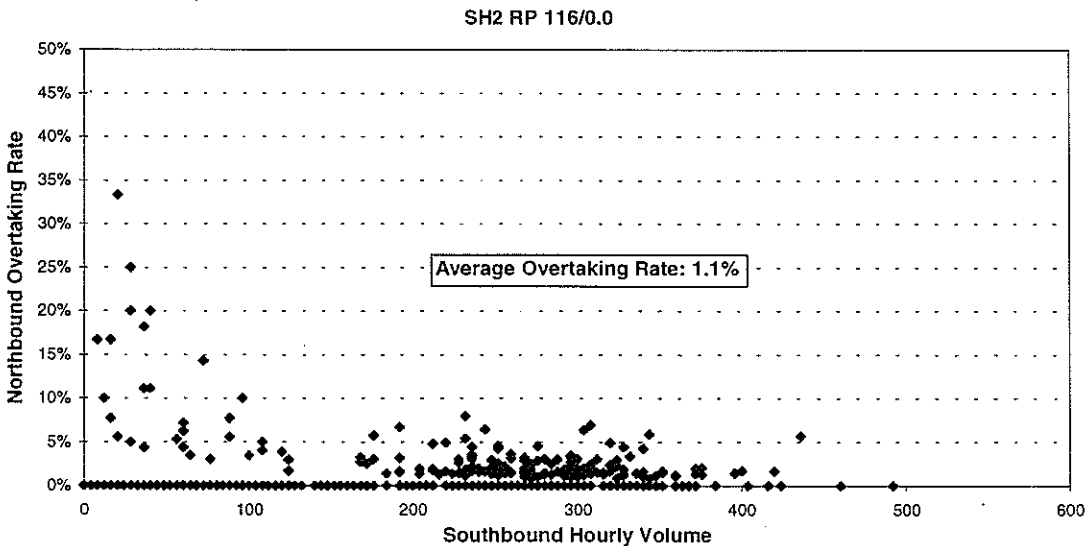


Figure 2.4 Overtaking rates (northbound) at Site 2/116, SH2 north of Katikati.



The SH3 site (Figures 2.5, 2.6) illustrates quite significant differences in overtaking rates in each direction. The eastbound traffic exhibits by far the strongest overtaking rates of all sites, while westbound traffic has the lowest. This may be a consequence of the Manawatu Gorge being immediately before the site for eastbound traffic, causing a build-up in vehicle platooning and driver frustration.

A comparison of prior passing opportunities in the 5km before each site was made to see whether a correlation with overtaking rates could be identified. For the purpose of this exercise, the proportions of highway with sight distances greater than 450m or 330m were determined. Apart from the above-mentioned site however, no clear trend was discernible between average overtaking rates and passing demand.

Figure 2.5 Overtaking rates (eastbound) at Site 3/491, SH3 near Woodville.

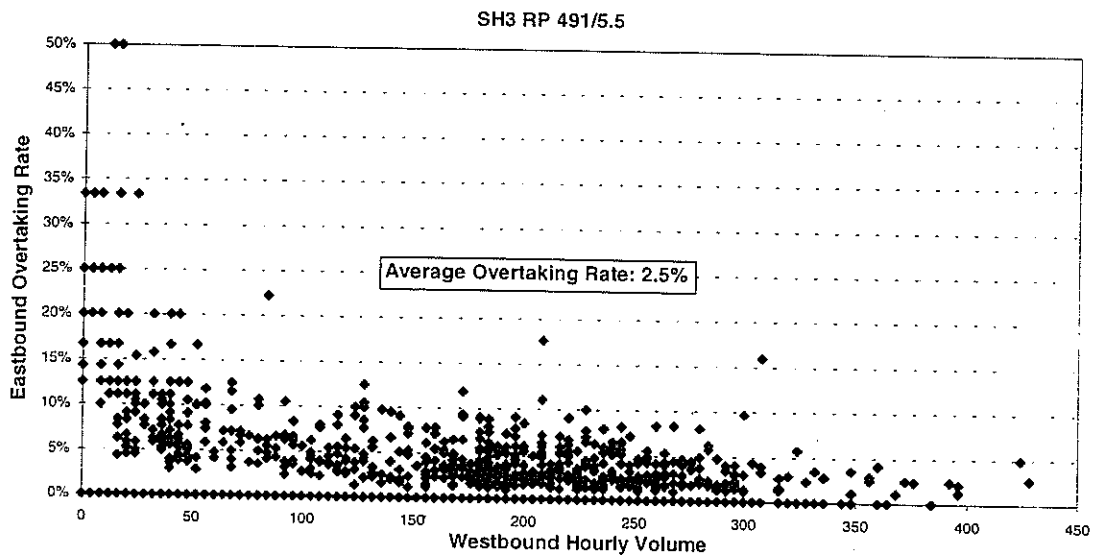
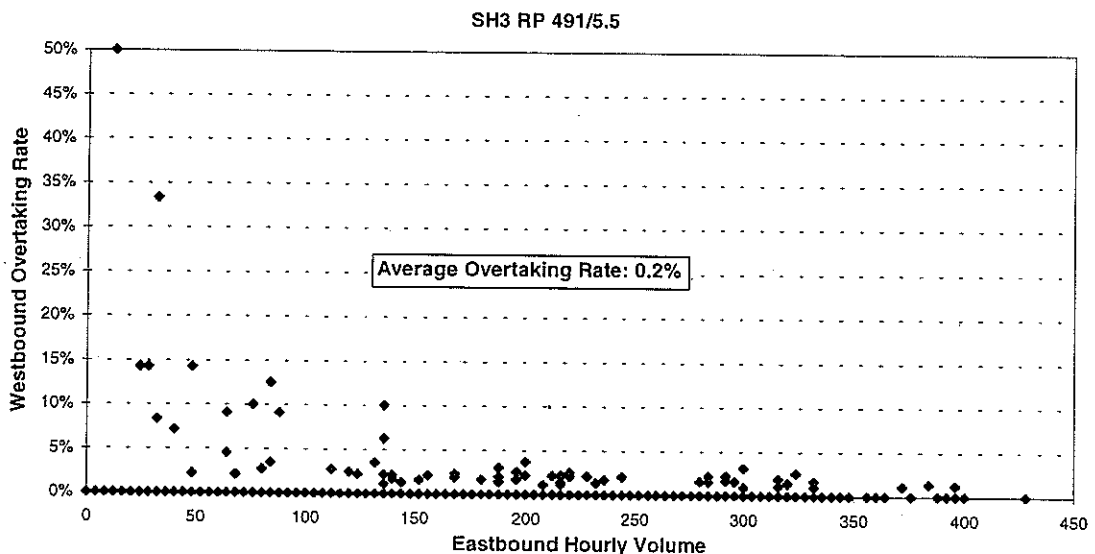


Figure 2.6 Overtaking rates (westbound) at Site 3/491, SH3 near Woodville.



Each of the plots indicates a generally decreasing relationship between overtaking rate and opposing traffic, similar to a 1/x or negative exponential function. This is a little deceptive because of the often-high number of 0% readings plotted on top of each other. However the results do tend to suggest that the maximum potential overtaking rates do indeed fall away with increasing volume.

Figure 2.7 Overtaking rates (westbound) at Site 58/0, SH58 near Pauatahanui.

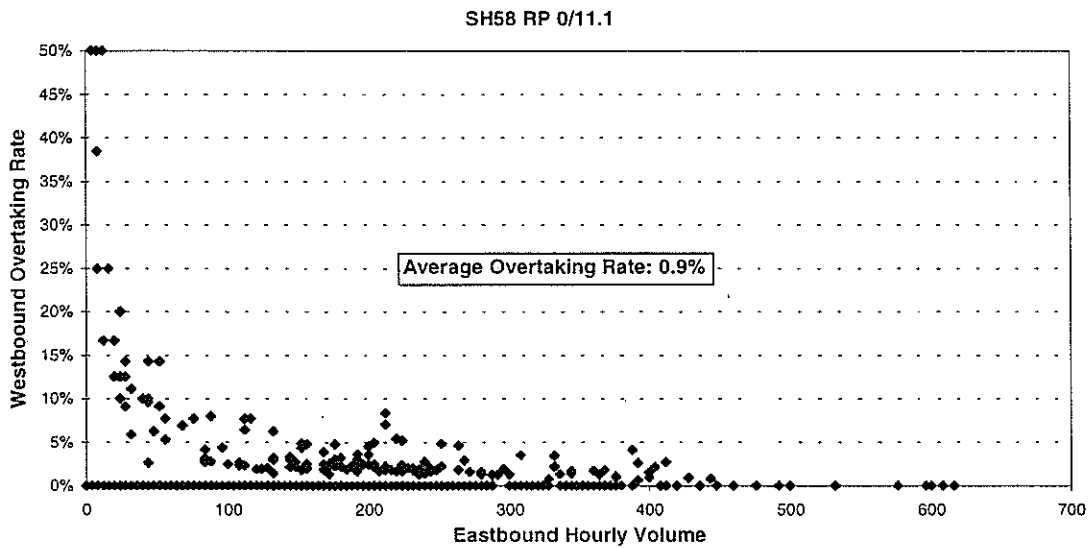
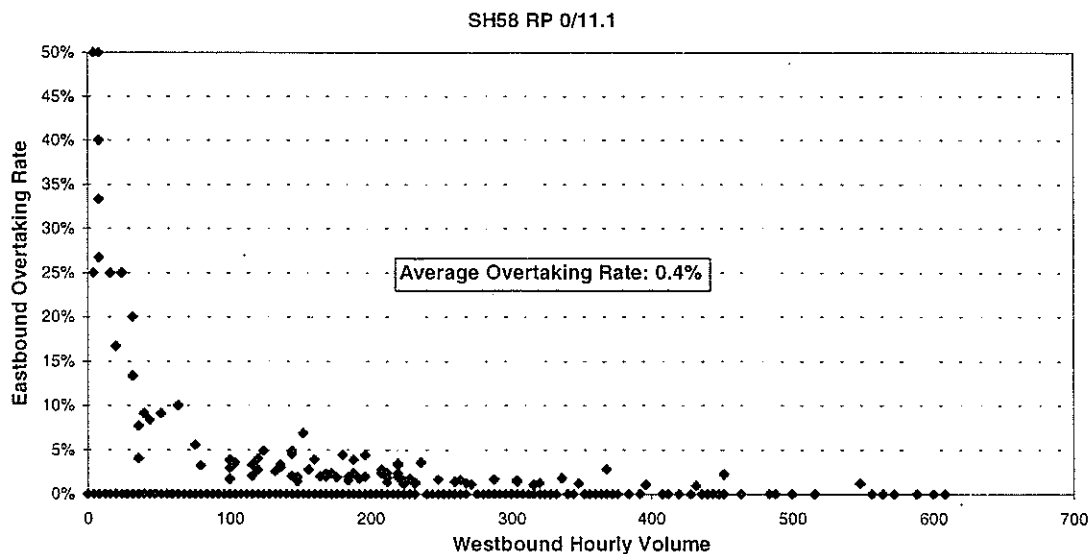


Figure 2.8 Overtaking rates (eastbound) at Site 58/0, SH58 near Pauatahanui.



The trends plotted are in line with the Percent Available Gaps (PAG) calculation currently used in the Transfund simplified passing lane evaluation, which is used to determine the supply of passing opportunities:

$$PAG = e^{-(0.008 \times \{One\text{-way Volume}\})} \quad (1)$$

Although plots could also be derived for overtaking rate as a function of traffic flow in the same direction, these are not as informative. First, because there is a direct relationship between traffic flow and overtaking rate in the same direction, the plot at low volumes forms patterns of parallel curves (e.g. at 4 veh/hr, the only possible overtaking rates are 0%, 25%, 50%, or 75%). Second, as discussed above, traffic flow in the same direction directly affects the demand for overtaking but this is not always matched by the available supply of overtaking opportunities. It is the supply that ultimately determines the actual amount of overtaking that occurs, and the opposing flow and alignment determine this.

2.2.4 Further Surveying

Subject to the results from the initial surveys, it was hoped to re-survey the sites after they were re-marked using the alternative horizontal curve warrant criteria. At the sites chosen, this would have had the effect of introducing no-overtaking lines where currently there were none. On the basis that no other aspect of the site had been changed, this would have addressed the question of whether drivers complied with the stricter markings.

A number of concerns arose about the usefulness of this exercise. For example, two of the sites were affected by adjacent works during or after the initial surveys. As mentioned above, the SH2 site had roadworks nearby while it was being surveyed. Although it does not appear to have affected the general pattern of overtaking rates with volumes, it is likely to have affected the overall level of overtaking so as to render a comparison survey invalid. Similarly, the SH58 site was affected by subsequent road works at the Pauatahanui bridge. The associated road realignment extended further than expected, to the curve at the eastern end of the survey site. This had the effect of changing the approach sight distance and probably the level of bunching prior to the site.

The relatively low level of overtaking (0%-2.5% of all vehicles on average) also raises questions about whether a significant enough trend would be detected as a result of marking of no-overtaking lines. Although over 20,000 vehicles were recorded at each site in each direction (and twice that at the SH3 site, which had been surveyed for two weeks), initial analysis suggests that it would still be very difficult to get an adequate level of change. If it were assumed that the overtaking rate approximated a binomial distribution, then a relatively small reduction in overtaking overall would be statistically significant. For example, a 0.5% overtaking rate over 20,000 vehicles requires about a 0.1% (absolute) change to be statistically significant at the 95% level. However comparison of the SH3 site data on a day-to-day basis revealed standard deviations that were 3 to 5 times that of a binomial distribution, requiring correspondingly large changes to be significant. A discernible trend when compared with traffic volumes is even less likely to be evident, given the previous distributions.

While in an ideal world, the effect of such marking would be to produce no overtaking, a number of drivers are likely to ignore or not notice the markings and base their decision on the visible road space in front of them. Thrush et al. (1998) noted that approximately 7% of overtaking-related crashes occurred at locations

marked with no-overtaking lines, while a more recent analysis given at the start of this Section 2 suggested an even higher figure.

The remaining SH3 site could still have been re-marked and re-surveyed. In having the highest and lowest recorded levels of overtaking in opposite directions at the same site, this site provides an interesting contrast. However it was felt that re-surveying only one site would not do justice to the purpose of the exercise.

2.3 Desktop Analysis of Effects of No-Overtaking Criteria

As an alternative investigation, a desktop analysis exercise was conducted to assess the implications of applying no-overtaking markings New Zealand-wide, using combined horizontal and vertical criteria. State Highway Road Geometry data was used to analyse the likely effects on:

- Lengths of road required to be marked with no-overtaking markings;
- Safety, as a result of changes in numbers of attempted passing manoeuvres;
- Travel times, as a result of changed passing opportunities.

Note that, for this work, only the rural SD criteria were used, and major urban areas were excluded, while minor urban settlements along the road were analysed as part of the rural lengths.

2.3.1 Expected Lengths of No-Overtaking Markings

As with the previous work recorded in Section 2.1, RGTRA was used with the SH Road Geometry data to produce SD data. Because the full New Zealand road network is available on a single Opus database, the 1998 SH Road Geometry data was used for this work. Although some changes in road geometry have occurred since then (1998) as a result of realignments and shape corrections, the overall length involved is relatively minor.

Again, offsets of 1000m (vertical-only SD) and 5m (combined horizontal/vertical SD) were used. In practice there are often situations where visibility is possible across a horizontal curve in open (rural) country. Therefore the true effect of a horizontal/vertical criterion is likely to be overstated. Manually identifying and adjusting for the actual location of all obstructions would be very time-consuming. However, some data was available from another project where this was done over a small section of the highways studied in this project, and this is compared later.

For simplicity of processing, only the four longest state highways (SH1N, 2, 1S, 6) were analysed, and in this way over 7200 lane-km were covered. These four highways have sufficient variety of terrain and alignment types to be considered fairly typical of the national roading network. The highways were split into smaller sub-lengths, as detailed in Table 2.3. This splitting also made it possible to avoid the major urban areas where extensive divided carriageways are prevalent.

Table 2.3 Breakdown of longer state highways into shorter lengths.

SH	Sub-section	From RS	To RS	Description
1N	1NY	1	303 (end 288)	Kaitaia – Orewa
	1NZ	478	969 (end 953)	Cambridge – Paremata
2	02X	0	146 (end 130)	Pokeno – Tauranga
	02Y	164	638 (end 626)	Mt Maunganui – Bay View
	02Z	661	946 (end 931)	Clive – Upper Hutt
1S	1SX	0	317 (end 311)	Picton – Woodend
	1SY	350	700 (end 683)	Hornby – Dunedin
	1SZ	729	920 (end 900)	Mosgiel – Invercargill
6	06X	0	116 (end 99)	Blenheim – Nelson
	06Y	130	639 (end 626)	Richmond – Fox Glacier
	06Z	639	1168 (end 1157)	Fox Glacier

After batch running of RGTRA analyses, the data was imported into an Access database for compilation. For each SD criteria, the total lengths of the network not meeting minimum SD levels were tabulated. Table 2.4 shows the results from both the existing (vertical SD only) requirement, and the alternative (both horizontal/vertical SD) requirement.

Table 2.4 Effect of different SD criteria on no-overtaking marking requirements.

SH	Total Lane Distance (km)	No-OT Dist. (Vt Only) (km)	% Vt Only SD	No-OT Dist. (Both Hz/Vt) (km)	% Both Hz/Vt SD	Relative Increase *
02X	290.1	77.3	26.6	189.5	65.3	2.45
02Y	937.3	194.3	20.7	696.2	74.3	3.58
02Z	503.4	76.9	15.3	300.2	59.6	3.91
06X	227.3	45.6	20.1	170.8	75.1	3.74
06Y	997.5	271.6	27.2	711.4	71.3	2.62
06Z	1055.2	256.4	24.3	727.9	69.0	2.84
1NY	593.2	169.1	28.5	470.7	79.3	2.78
1NZ	965.8	172.2	17.8	555.4	57.5	3.22
1SX	630.3	123.7	19.6	504.6	80.1	4.08
1SY	681.6	93.5	13.7	249.0	36.5	2.66
1SZ	381.4	82.2	21.5	221.3	58.0	2.69
Total	7263.0	1562.7	21.5	4796.9	66.0	3.07

No-OT Dist. – no-overtaking distance; Vt – Vertical; Hz – Horizontal

* Increased length of markings relative to existing vert. SD-only criterion.

This table shows that a change to a criterion involving both horizontal and vertical SD restrictions would produce a significant increase in the amount of no-overtaking lines marked, being approximately three-fold. Conversely the figures can also be interpreted as a >50% reduction in lengths having adequate passing SD. At this broad scale, no discernible trend is evident that shows how routes in different terrain are affected by the change in criteria.

As discussed above, if combined horizontal/vertical criteria are used they are likely to be unduly restrictive in places where drivers can see across corners that have no obstructions. To check the impact of this theory, a quick analysis was done on a

section of SH1S between Blenheim and Kaikoura. This section was recently the subject of a passing lane study undertaken by Opus, so data were available on obstructions along the route and sight distances had been calculated on this basis. An additional RGTRA run, using the slightly less restrictive offset of 10m each side, was also carried out and analysed. The table below compares these data with the data calculated above.

Table 2.5 Effect of obstructions on no-overtaking marking requirements.

Criteria	No-OT Dist. (km)	% of SH Marked	Relative Increase*
All of SH1S Blenheim-Kaikoura	269.2	–	–
Existing Vert SD Only (1000m offsets)	60.5	22.5	1.00
Combined Horiz/Vert SD (5m offsets)	214.5	79.7	3.54
Combined Horiz/Vert SD (10m offsets)	88.8	33.0	1.47
Adjusted Horiz/Vert SD (actual offsets)	120.0	44.6	1.98

* Increased length of line markings relative to existing vert. SD-only criterion.

It is apparent that the 5m offset used for assessing the combined horizontal/vertical criterion is rather extreme when compared with more realistic offset data. On the other hand, using a 10m offset instead would be too conservative. The picture is complicated by the fact that different highway sections will produce different comparisons. But the data above suggest that the expected increase in no-overtaking markings might be more like a two-fold increase, rather than the three-fold increase suggested previously using the 5m offsets.

A caveat on the above findings is that the tabulation involved a simple assessment of each 10m segment as to whether it met the minimum SD requirement. The actual criteria specify minimum continuous lengths of road that are required before lines are marked. Therefore isolated sub-standard segments would not be marked but would be recorded in the above tallies. In practice, sight distances are not erratic, tending to slowly fall as an obstruction is approached, so isolated sub-standard segments are rare. This is balanced a little by the sections of no-overtaking lines, near enough to each other to be marked continuously through, that would not be reflected in the above tallies.

Another point to remember is that this broad-based analysis does not consider any local effects of small towns, side roads, opposing passing lanes, or narrow bridges, on overtaking provisions. Irrespective of which SD criteria are used, all these features are likely to decrease the effective amount of reasonable overtaking areas.

2.3.2 Assessing the Effects on Safety & Efficiency

An increase in no-overtaking markings is likely to limit the amount of overtaking available, with a probable increase in travel times. Conversely however, the reduced overtaking may produce a reduction in unsafe manoeuvres and a corresponding reduction in crashes. To test this hypothesis, rural road simulations were run using both vertical-only and combined vertical/horizontal criteria for a range of terrain and traffic conditions.

The scenarios were tested using ARRB Transport Research's TRARR 4 modelling package. This software is designed to simulate vehicles travelling on rural roads and is generally used to test the effects of providing realignments and passing lanes. Key outputs include mean travel times and the number of overtakings completed.

In defining a road file for TRARR, the user can specify sections of road with no-overtaking (or "barrier") lines. By basing these sections on the different marking criteria, different road files can be used to compare the effect of increased no-overtaking lines. TRARR's standard no-overtaking line is designed so that no vehicle infringes on it. However, an "intermediate" level of no-overtaking line allows some vehicle types (e.g. "sporty cars") to be allowed to overtake where other passing criteria are satisfied. This would reflect what in reality often happens on the road. Simple sensitivity testing of the effects of no-overtaking compliance rates on outputs was checked, and is discussed after Figure 2.11. Initially however, all vehicles were assumed to comply with the markings.

Typical "flat", "rolling" and "mountainous" terrain road sections were created for modelling vehicle interactions using TRARR. The highway data selected was used in previous research into passing demand (Koorey & Tate 1997). Each of the three terrain types were defined by average values for gradient (in terms of total rises/falls per km, or "hilliness") and curvature (in terms of degrees per km, or "bendiness"), using criteria specified by McLarin (1997) and based on the *Project Evaluation Manual* definitions (Transfund 1999). Highway sections that met these criteria were taken from a database of New Zealand State Highway Road Geometry (Cenek et al. 1997). Road sections of 50km length were created for each terrain, comprising ten 5-km sections from various state highways that had the most appropriate levels of "bendiness" and "hilliness". In each case the overall average gradient and curvature values were very close to the specified values. No-overtaking lines were inserted in the road files, based on sections that did not have at least 330m of clear sight distance, depending on the given criteria.

A range of traffic volumes (between 50 and 1000 veh/hr two-way) was simulated to determine outputs as a function of volume. A "typical" heavy vehicle composition of 10% was used throughout. Initial bunching levels (i.e. the proportion of vehicles following others) were set that increased with volume (based on findings from another part of this research project) ranging from 5% to 59%. Simulation times were adjusted for each volume so that approximately 2000 vehicles (two-way) were modelled and analysed in each case.

All up, 54 basic combinations were simulated using TRARR (9 volumes \times 2 SD criteria \times 3 terrains), plus some additional runs that were made to check the sensitivity of changing no-overtaking compliance. Batch routines were used to automate the running and compiling of the simulations, with the final results analysed in Excel.

Figure 2.9 shows the effect on overall mean travel times (over the 50km analysis lengths) of changing SD criteria. Comparisons by volume and terrain are given with trend lines fitted to the points in each series. As would be expected, there is a clear

trend towards increasing travel times with either rougher terrain or higher traffic volumes. However, a change from vertical-only to combined horizontal/vertical SD criteria increases mean travel times by up to 9%. For flat, rolling, and mountainous terrain the averages are 2.7%, 5.3% and 5.8% respectively. The greatest effects are found with traffic volumes of 150-300 veh/hr; higher volumes tend to be more constrained by other traffic irrespective of actual passing opportunities.

Figure 2.9 Mean travel times with changing SD criteria.

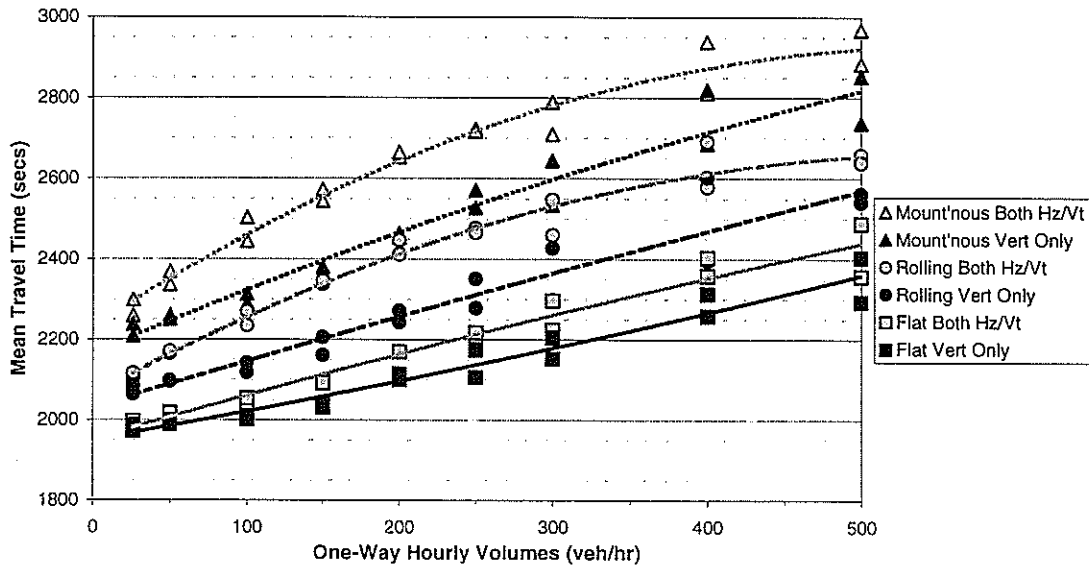
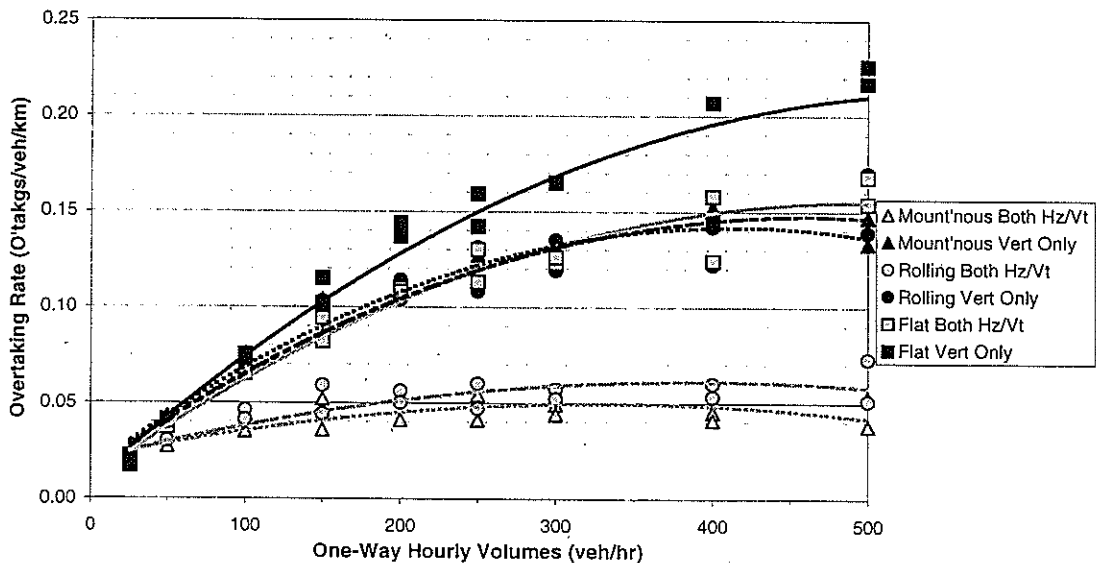


Figure 2.10 (using same conventions as in Figure 2.9) shows the average overtaking rate for each situation. Overtaking rate is defined as the number of successful overtakings per vehicle per kilometre travelled. For example, with a rate of 0.10 along 3km of road, 200 vehicles/hr would be expected to complete 60 ($= 0.1 \times 3 \times 200$) overtakings per hour.

Figure 2.10 Average overtaking rate with changing SD criteria.

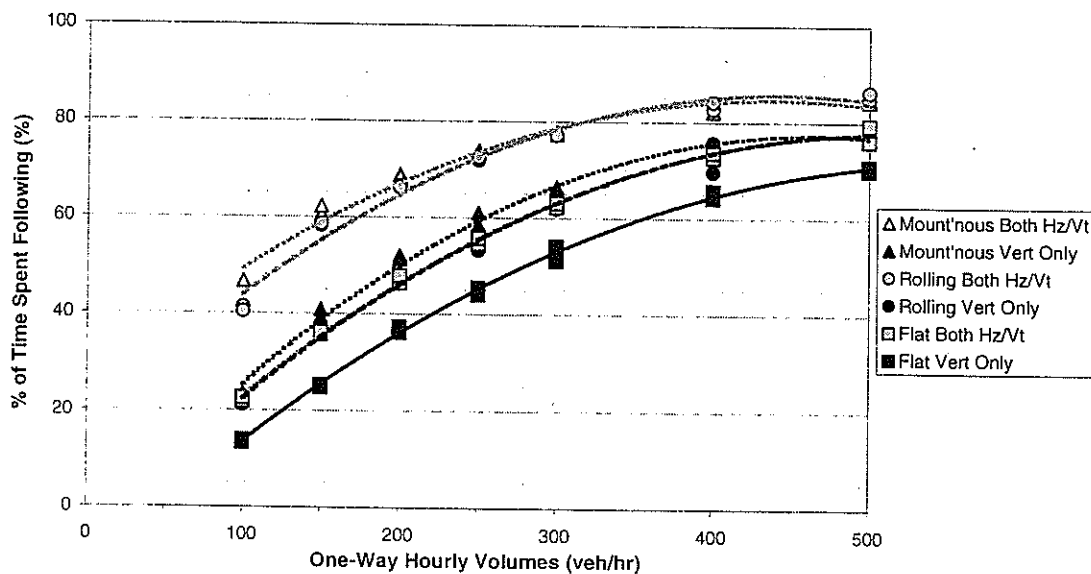


The plots show that overtaking rates tend to initially increase with volume until opposing traffic limits the ability to overtake. Likewise, more rugged terrain limits overtaking opportunities, although it is interesting that little difference between results for rolling and mountainous terrains is shown. A change in SD criteria significantly reduces overtaking rates, particularly for the rolling and mountainous terrains. Flat terrain rates reduce by about 20%, while for other terrains the reduction is at least 50% and also it is more noticeable with increasing volumes.

Not directly evident from this is the effect that the reduction in overtaking has on safety. Intuitively however, the reductions above understate the improvement in overtaking safety. This is because many of the previous overtakings now not undertaken would have been of the more “dangerous” variety, i.e. those most likely to lead to crashes because they have occurred in unsuitable locations.

Another way to consider the effect on safety and efficiency is to look at driver frustration as a result of more time spent following other vehicles. Figure 2.11 (using the same conventions as in Figure 2.9) shows how the percent of time spent following (PTSF) changes for each situation. Note that, as TRARR does not always accurately model PTSF for low volumes with long simulation times, results below 100 veh/hr have been excluded.

Figure 2.11 Percent of time spent following (PTSF) with changing SD criteria.



As expected, PTSF generally increases with volume and more rugged terrain (although there is little distinction between rolling and mountainous terrain at high volumes). A change to a combined SD criterion results in an average 20-30% increase in PTSF, with greater effects at lower volumes. This will impact in terms of driver congestion benefits, as outlined in Transfund’s *Project Evaluation Manual* Appendix A4, with a subsequent increase in travel time costs. The increase in PTSF is also likely to reflect an increase in aggressive manoeuvres by drivers who are unable to find sufficient passing opportunities. This will somewhat offset the

estimated savings predicted from the above overtaking rate analysis, although the net effect is still expected to be a significant reduction in overtaking-related crashes.

Given the known proportions of the national road network with each category of terrain and volume, the above findings could be scaled up to represent likely findings for the whole country. Although the changes in overtaking rates and, to a lesser extent, PTSF are significant with a move to a combined horizontal/vertical criteria, the impact on overall travel times is not as substantial.

2.3.3 Assessing the Effects on Driver Compliance

The above TRARR simulations are based on the assumption that all drivers obey the marked no-overtaking restrictions. In reality there is likely to be a minority of drivers who choose to ignore the markings and rely on their judgement, particularly if they feel they have a vehicle with greater-than-average acceleration. TRARR recognises this by means of a "compliance" parameter to specify which vehicle types modelled will obey the markings. Normally four vehicle sub-classes (representing different "types" of driver) making up 33% of the car stream are set to ignore markings by default in the simulations.

To assess the sensitivity of the above results to this scenario, a repeat run of TRARR simulations was done at 500 veh/hr (two-way) using the default TRARR compliance parameter values. In theory the effect of this should be to increase overtakings and reduce travel times. The net result however was not very significant: overtakings increased by no more than 9.8% and this only translated into mean travel time reductions of no more than 1.1%. In fact, in a couple of the simulations the results were slightly worse than previously. When looking at the effect of driver compliance on the change between vertical criteria and horizontal/vertical criteria, the resulting changes were very similar to the findings described in Sections 2.3.1 and 2.3.2.

A reasonable theory could be that driver compliance would get worse with the introduction of a stricter and more widespread no-overtaking criterion, especially without any equivalent increase in enforcement. However, even changing from a vertical SD criteria with strict compliance, to a combined SD criteria with moderate compliance, produces a very similar result to the previous plots (Figures 2.9-2.11). Therefore, unless actual driver compliance is in fact far worse than modelled, it is not a significant factor when considering alternative no-overtaking marking criteria.

For comparison, a review was made of overseas studies that have investigated driver compliance. Lyles (1982) tested different combinations of no-overtaking markings and signs (commonly used to denote no-passing zones in the US) for driver compliance. This was achieved by driving along selected lengths back and forth at a constant speed and observing overtaking vehicles. Lyles found that almost half of all passing manoeuvres started in no-passing zones when only pavement markings were present, although only 30% when unopposed by oncoming vehicles. Compliance improved noticeably when signs were also present, with 20%-40% not complying (10%-20% when unopposed).

Bacon et al. (1963) analysed over 1300 self-completed questionnaires on passing behaviour in Michigan, where both markings and signs are used. He found that 14% of drivers had overtaken completely in no-passing zones, although 5% stated “only in rare cases”. More than 40% had also started overtaking before the end of a no-passing zone.

Although these studies are not entirely comparable with New Zealand road conditions, given the differences in signing, marking and general availability of passing opportunities, they indicate that non-compliance of no-overtaking markings can be a significant effect. They also indicate that modelling with the TRARR default values probably provided more typical indications of the true effects.

2.4 Discussion

There is reluctance in New Zealand to mark every section of highway limited by poor horizontal SD for at least two reasons. First, drivers are assumed to be better able to identify that an oncoming horizontal curve will obstruct their sight distance, whereas many vertical curves are more subtle. Second, the concern is that in marking every technically required no-overtaking section, the net effect of the markings will be diluted and lead to poorer compliance overall. Both these arguments need further examination.

The fact that New Zealand policy makes the distinction between vertical and horizontal sight distances however, is evidently a cause for much concern nationwide, as evidenced by the findings of a recent survey on no-overtaking line standards (LTSA 2000). As well as horizontal curves, Road Controlling Authorities interviewed in the survey had concerns about current eye and object heights used, consideration of different speed environments, and the level of discretion allowed for marking no-overtaking lines.

Recent sight distance theory has introduced the concept of “Intermediate Sight Distance” (ISD). This distance is less than the computed sight distance required for overtaking, but better reflects when drivers will consider overtaking. ISD is defined as twice the stopping sight distance, which thus allows a person attempting to overtake sufficient time to abort an overtaking manoeuvre should an oncoming vehicle appear. ISD varies with travel speed, and Table 2.6 lists rounded ISDs for various speeds, assuming a reaction time of 2.5 seconds and a level grade.

It is instructive to compare these values with the current rural no-overtaking visibility criterion of 330m. On high speed alignments this constant distance may not be sufficient, while in very low speed alignments it is probably too conservative. Given the research mentioned at the start of this Section 2, that highlighted drivers’ difficulty in assessing approach speeds and relating them to gap acceptance, more flexible no-overtaking guidance using ISDs is probably sensible. As well as providing safer high-speed locations, limiting the length of no-overtaking markings in low-speed environments may also help with acceptance and compliance of these markings.

Table 2.6 Intermediate sight distances (ISD) at different speeds.

Travel Speed (km/h)	Rounded ISD (m)
50	130
60	170
70	220
80	270
90	340
100	410
110	500
120	570

The overtaking rate surveys were carried out in areas where insufficient safe overtaking sight distances are available for most vehicles. (Although based on 330m criteria, the surrounding speed environments also gave similar ISDs.) The surveys did not identify whether the overtaking that was recorded was a result of poor or risky judgement by the drivers, more powerful overtaking vehicles, or slower than normal overtaken vehicles (as the latter two situations were likely to be safe in many cases). Certainly all three situations are likely to have occurred and will continue to do so even if markings are changed.

Situations, like the sites surveyed, highlight a need to distinguish between the isolated horizontal curve and the continuously curved, winding environment. The latter probably needs no markings to restrict overtaking because the sight distance never increases sufficiently for drivers to consider the possibility of overtaking. With isolated curves however, the approach section may provide an adequate opportunity to overtake. The critical information to convey to the driver is where along the approach does the remaining clear distance fall below the safe overtaking distance. Given the higher speeds likely on many approaches, the information is even more significant.

A short straight, such as those surveyed, may not meet the overtaking criteria at all but its appearance may suggest otherwise. If the straight is in the middle of an otherwise winding environment, drivers may also be more tempted to try to use it to overtake. This situation probably warrants marking no-overtaking lines where the length does not meet the ISD, yet does not fall far short. One possible criterion could be to mark sites where the available sight distance is between half the ISD (equivalent to the stopping sight distance) and the ISD. For example, a short 300m straight in a 90-km/hr speed environment would be marked fully with no-overtaking markings. If the straight was 600m long, only the last 340m would be marked and not necessarily right to the curve. If the straight was only 150m long, no marking would be deemed necessary.

Allowing no-overtaking markings near horizontal curves in circumstances such as those mentioned above may provide a suitable compromise to increasing their application fully at horizontal curves. As indicated by the RGTRA analysis, a full change to a combined SD criteria would increase the amount of markings by at least two-fold, if not higher. This approach would allow those sites most likely to cause problems for drivers to be targeted.

One problem with any set standard is that differently performing vehicles will require different overtaking space. In the UK, two tiers of sight distance warning are used: “prohibitive” and “advisory” no-overtaking lines. At the latter, drivers are allowed to overtake other vehicles at their own discretion. This may be a way of providing some varying feedback to drivers; for example by using white and yellow solid lines. In the same way that motorists tend to adjust signed curve advisory speeds to suit their vehicle, they can adjust their overtaking behaviour accordingly. Alternatively this may also be reasonably done by the current use of advance no-overtaking warning lines.

To improve compliance with more restrictive no-overtaking criteria, the use of signs similar to those used overseas should be investigated. For example, common signs used in the US are rectangular “DO NOT PASS” signs and triangular “NO PASSING ZONE” pennants. While their presence is likely to improve compliance, this would have to be weighed up against the additional cost of their installation.

Given that the most hazardous situations are likely to occur with greater traffic volumes, no-overtaking criteria that take into account traffic flows may be prudent to consider as well. Drivers are likely to choose gaps partly based on their recent experience of opposing traffic, affecting the level at which no-overtaking areas should be highlighted. It may be that, above a certain traffic volume, no overtaking is permitted on two-lane highways except at formal passing opportunities. Certainly, this approach has been taken on some high-volume highways in New Zealand already (e.g. SH1 Mercer – Huntly, SH1 Pukerua Bay – Paekakariki).

While the above discussion has focused on two-lane sections, the implications also need to be applied to other situations where no-overtaking markings are used (or could be used). Some key situations that come to mind are when encountering an opposing passing lane, approaching either side of major intersections, and approaching narrow bridges. Specific studies would be needed to relate driver behaviour with and without no-overtaking markings present to the respective crash records in these situations.

Concerns about compliance also raise the question of effectiveness of no-overtaking markings. For example, if crashes occur where drivers are currently overtaking, and yet the site does not, technically, warrant installing no-overtaking lines, putting in no-overtaking lines because of a recommendation from a crash study will not necessarily stop drivers from overtaking.

Changes such as those discussed above are not taken lightly, given the potential safety implications of the revised markings and subsequent driver behaviour. One way to test options with some safety would be to use a driving simulator, such as that operated by Waikato University. This approach was used with some success for recent research into the effects of different passing lane markings on driver manoeuvres (TERNZ 2001).

3. Field Calibration of Simplified Procedures

Further refinements to the existing simplified procedures (Transfund 1999) for evaluating passing lanes, are considered here in the light of field surveys, desktop analysis and new research.

3.1 Background

Koorey et al. (1999) investigated the development of a simpler system to determine the need for, location of, and benefits to be derived from, passing lanes. Part of the research also looked at measures of driver frustration resulting from inadequate passing opportunities.

A tangible Willingness To Pay (WTP) for passing opportunities related to driver frustration was determined. A simplified model for assessing the optimum location of passing lanes was also developed. The model is based on comparing the supply of and demand for passing opportunities along a route and requires less input data and analysis time than TRARR.

The driver frustration WTP and methodology for the simplified model were incorporated in the Transfund *Project Evaluation Manual* (PEM 1999). This enabled simple passing lane projects to be evaluated more easily, while also allowing all passing lane projects to claim additional benefits by reducing frustration. Some subsequent minor changes have been made to the procedures in light of feedback from consultants and Transit New Zealand.

However, a number of problems requiring further work remain:

- Although WTP was found to be significantly correlated with the easily measurable “Unsatisfied Passing Demand”, further work was required to refine the relationship. In the interim, a constant WTP value has been prescribed that, while providing a significant quantum of benefits, does not vary results according to actual passing demand at the site. An alternative WTP measure using the “Percent of Time Spent Following” measure from TRARR outputs has also been provided, however it has not been validated against the research data.
- The simplified model relies on knowing the maximum practical overtaking rate on a road section that allows clear passing. Although a theoretical value was derived from first principles, field surveys are required to confirm this value. The model also assumes that there is an Accrued Passing Demand (APD) at the start of the analysis length. Although a means of determining initial APD from bunching field data (i.e. recorded levels of vehicles following others) has been introduced, it is not clear how changes in APD relate to changes in bunching down the road.

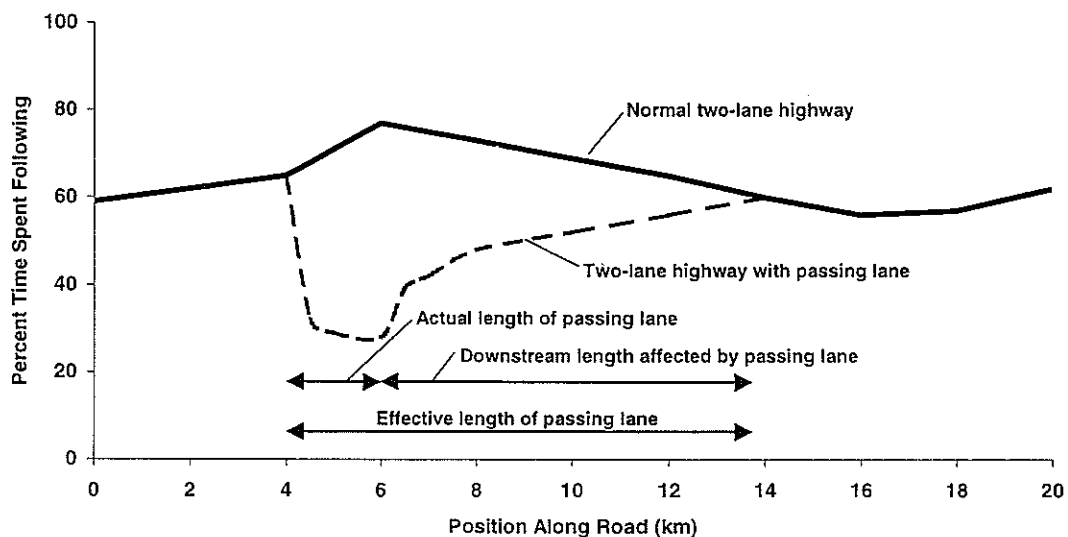
Investigation of these issues is discussed below. Readers unfamiliar with the existing simplified procedures should refer to Appendix A10 of the PEM.

3.2 Highway Capacity Manual Analysis Procedures for Passing Lanes

The 2000 revision of the US Highway Capacity Manual (HCM, TRB 2000) has made major changes to the way that two-lane highway segments are analysed. This is the result of a major research programme undertaken to determine level of service effects and to incorporate passing lane segments more accurately (Harwood et al. 1999). The findings are based on the results of simulation runs (using the US TWOPAS model), comparing the variations of bunching, and speeds with and without the presence of a passing lane. It is instructive to compare the US approach taken with the existing simplified method for analysing passing lanes in New Zealand.

The revised approach presents a specific operational analysis procedure for passing lanes, something that was lacking in earlier HCM versions. Passing lanes generally decrease PTSF and increase Average Travel Speeds (ATS). Figure 3.1 illustrates the typical effect of a passing lane on PTSF, relative to position down the highway. A similar but inverse relationship can be shown for ATS.

Figure 3.1 Example of the operational effect of a passing lane on percent time spent following (PTSF)
from Harwood et al. 1999.



Analysis of the above can be simplified by dividing the analysis length into four regions.

- Upstream of the passing lane (length L_u);
- Within the passing lane, including tapers (length L_{pl});
- Downstream of the passing lane but within its effective length (length L_{de});
- Downstream of a passing lane but beyond its effective length (length L_d).

Figure 3.2 shows the resulting simplified relationship for PTSF, and Figure 3.3 does likewise for ATS (both figures from Harwood et al. 1999).

Figure 3.2 Effect of a passing lane on percent time spent following (PTSF) as represented in the operational analysis methodology.

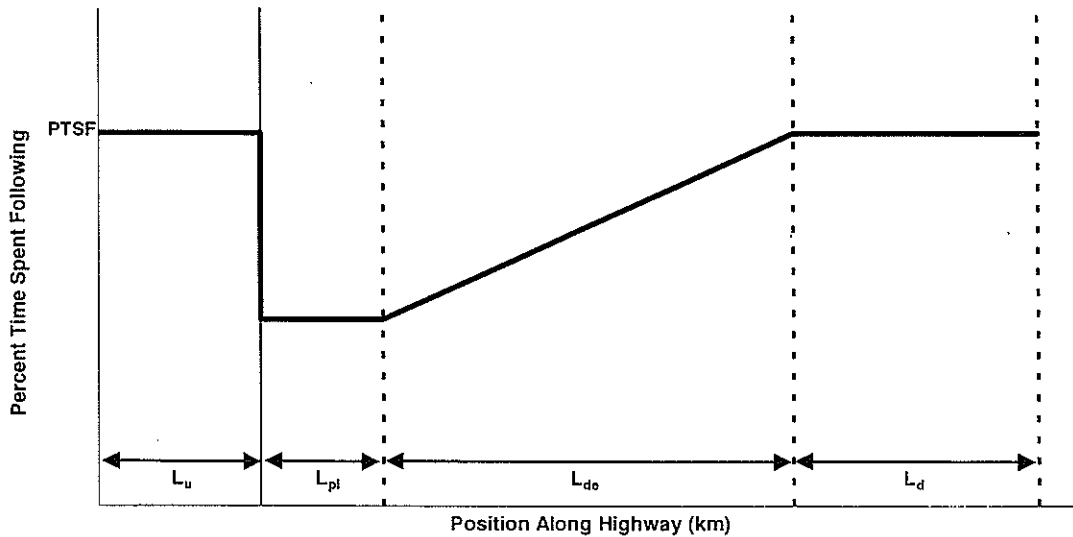
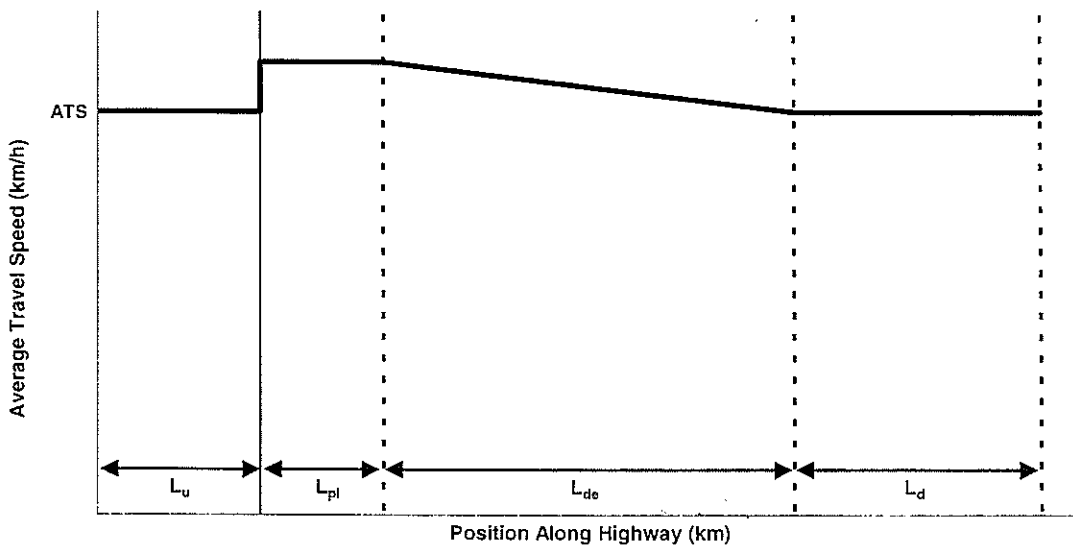


Figure 3.3 Effect of a passing lane on average travel speed (ATS) as represented in the operational analysis methodology.



The length of the segment upstream of the passing lane (L_u) and the length of the passing lane itself (L_{pl}) are readily determined when the location (or proposed location) of the passing lane is known. The length of the downstream highway segment within the effective length of the passing lane (L_{de}) is determined from Table 3.1. Note that the volumes are in terms of equivalent passenger cars per hour (pc/hr). The values represent the downstream distance required for PTSF or ATS with a passing lane in place to return to the values that would have existed if no passing lane were present.

Table 3.1 Downstream length of roadway affected by passing lanes on directional segments in level and rolling terrain.

One-Way Flow Rate (pc/hr)	Downstream Length of Roadway, L_{do} , affected (km)	
	PTSF (%)	ATS (km/hr)
200 or less	20.9	2.8
400	13.0	2.8
700	9.1	2.8
1000 or more	5.8	2.8

Source: Harwood et al. 1999

pc – passenger car, PTSF percent time spent following; ATS average travel speed.

Interestingly, the length of downstream highway likely to be affected in terms of ATS does not vary with volume. The relative reduction in PTSF or increase in ATS at the start of the passing lane can then be determined from Table 3.2.

Table 3.2 Estimation of relative change in PTSF and ATS within a passing lane.

One-Way Flow Rate (pc/hr)	Relative Change to Prior Highway	
	PTSF (%)	ATS (km/hr)
0-300	0.58	1.08
300-600	0.61	1.10
>600	0.62	1.11

Source: Harwood et al. 1999

pc – passenger car, PTSF percent time spent following; ATS average travel speed.

Using the above relationships, for example, a passing lane with 400 pc/hr one-way with a prior PTSF of 50% could be expected to reduce the PTSF to $50 \times 0.61 = 30.5\%$ over the length of the passing lane. The PTSF would then slowly climb back up linearly to 50% over the next 13.0 km. Similar results could be determined for the effect on travel speeds. The HCM procedure also contains simplified methods for determining the initial values of PTSF and ATS, depending on the traffic volumes and available sight distances.

Note that the above relationships are dependent on the studied passing lane being of optimal length, otherwise fewer benefits may be realised. Table 3.3 lists suggested lengths for different traffic volumes.

Table 3.3 Optimal lengths of passing lanes for different flow rates.

One-Way Flow Rate (pc/hr)	Optimal Passing Lane Length (km)
100	≤ 0.8
200	0.8 – 1.2
400	1.2 – 1.6
≥700	1.6 – 3.2

Source: TRB 2000

The above procedures are designed only for level or rolling terrain. For mountainous terrain or other significant upgrades, a similar analysis for uphill climbing lanes is provided, but with additional effects of grade and heavy vehicles taken into consideration.

The revised HCM procedures provide a fairly straightforward means of assessing the effects of a passing lane on vehicle bunching and travel speeds along a two-lane highway. It may be reasonable to consider their application for assessment of passing lanes in New Zealand. However, the analysis may be too simplified to be robust enough for specific project evaluation and funding.

Whether the results can be directly applied to two-lane roads in New Zealand is also questionable, although it is not likely that there are significant differences. Further investigation and comparison are necessary to confirm their appropriateness for evaluating passing lane projects in New Zealand, but the timing of their publication has not made it feasible to do so within this research. However their use as a tool for more strategic passing lane studies is probably valid.

3.3 Driver Frustration v Willingness To Pay

3.3.1 Existing WTP Derivation

The existing passing lane WTP factor was derived from the length of passing lane constructed (see Koorey et al. 1999 for more details about the original survey and analysis). For example, for the same traffic volume a 1600-m passing lane would derive twice as many benefits from reduced frustration as an 800-m passing lane. It could be argued that such benefits are likely to have diminishing returns as the length of passing lane increases. Likewise, the process does not appear to distinguish between similar passing lanes with similar traffic volumes that have different levels of passing demand (e.g. because of the surrounding passing opportunities). However, the original thinking underlying the chosen approach was that the WTP was valuing people's preference to have the availability of a passing lane, irrespective of their need to use it. The effects of passing lane length and passing demand on efficiency benefits would in theory be identified from travel-time savings.

However driver frustration benefits from passing lanes have been suggested to be better related to the reduction in time spent following (or "bunching"). This would have the effect of implicitly taking into account factors such as passing lane length and passing demand on earlier highway sections. As a result, WTP would probably vary for different passing lane sites, and for different traffic volumes at the same site (e.g. night v daytime flows).

In TRARR analyses, the PTSF is a direct output from each simulation. This allows us to compare its effect on WTP. For simplified passing lane evaluations without TRARR modelling, the equivalent measure can be determined using the PEM's congestion calculations (PEM Appendix A4), whereby saturation ratio (= volume/capacity), terrain type and percentage no-passing for the road section, allow us to determine "Percent Time Delayed" (PTD). This however is not a particularly straightforward exercise, and Transfund's current guidelines do not explain clearly how this approach applies to a passing lane situation. It may instead be more feasible to assess it from the APD calculations and this will be considered later

In the original WTP surveys, respondents were asked to value their WTP for hypothetically having continuous passing lanes along the route they had travelled. This was then related to actual known parameters for that road (such as traffic volume and available passing sight distance) used to determine Unsatisfied Passing Demand (UPD).

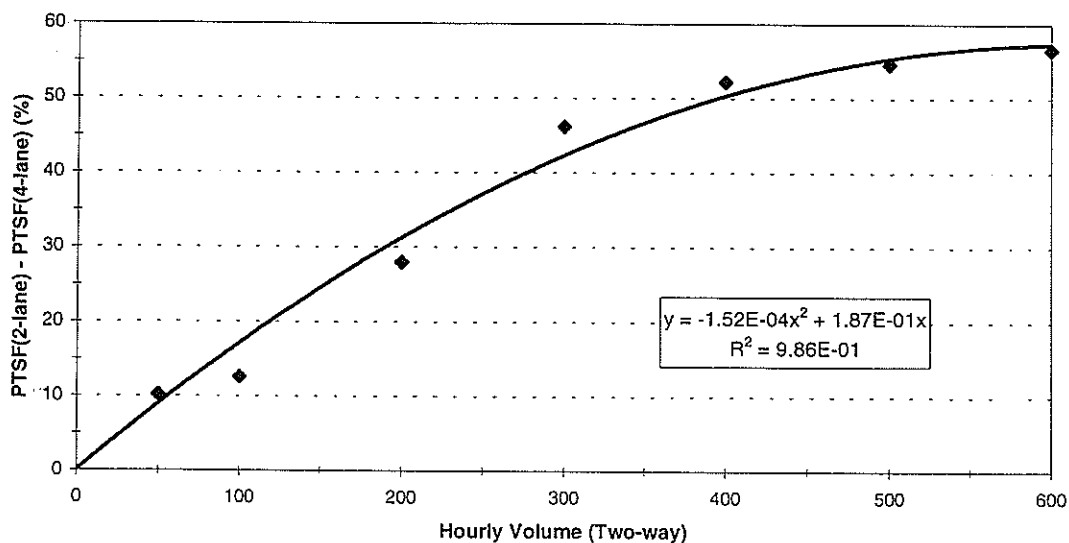
The proposed alternative approach is to look at the change in PTSF between the original (mostly two-lane) road and the hypothetical continuous passing lanes. These measures of PTSF could be determined using TRARR modelling of the original road lengths, with and without the passing lanes. The differences in PTSF could then be compared with the WTP values given by the respondents. The hypothesis was that increasing difference in PTSF, as a proxy for driver frustration, should result in increased WTP values. This could enable a quantification of the intangible benefits in providing passing opportunities on a particular section of road, where the traffic and terrain details were known.

3.3.2 Analysis and Discussion of WTP

Each of the road sections used in the original survey was created as a ROAD file for TRARR modelling. Where known, existing passing lanes were inserted to reflect the available passing opportunities at the time of survey. Another copy of each ROAD file was also made, with continuous passing lanes (i.e. undivided four-laning) throughout. Both sets were modelled against a series of traffic volumes between 50 and 600 veh/hr; and this covered the range of daytime volumes found at the sites.

Some experiences with TRARR suggest that it is not particularly good at modelling continuous four-laning, especially at high traffic volumes. In the current research however, the traffic volumes were generally low (<40% of capacity) and only the relative difference between the two sets was of interest, rather than the absolute performance measures. Tate (1995) observed that, even when uncalibrated, TRARR still performs quite adequately in determining relative changes in output measures.

Figure 3.4 PTSF relationship for Taupo-Turangi section of SH1.



For each road, the differences in PTSF between the existing and continuous passing lane cases were plotted against traffic volume to determine a relationship. Figure 3.4 shows one of the typical relationships derived. Note that the shape mirrors that of standard PTSF-volume relationships (e.g. Figure 2.11), supporting its validity.

The relationships were then used to determine a likely change in PTSF for each of the respondents from the original survey. This was based on their route chosen and their assessment of traffic flow while they travelled.

The individual payments for time and distance were plotted against the difference in PTSF (Figures 3.5 and 3.6).

Figure 3.5 WTP (Time Values) for reduction in PTSF (% time spent following).

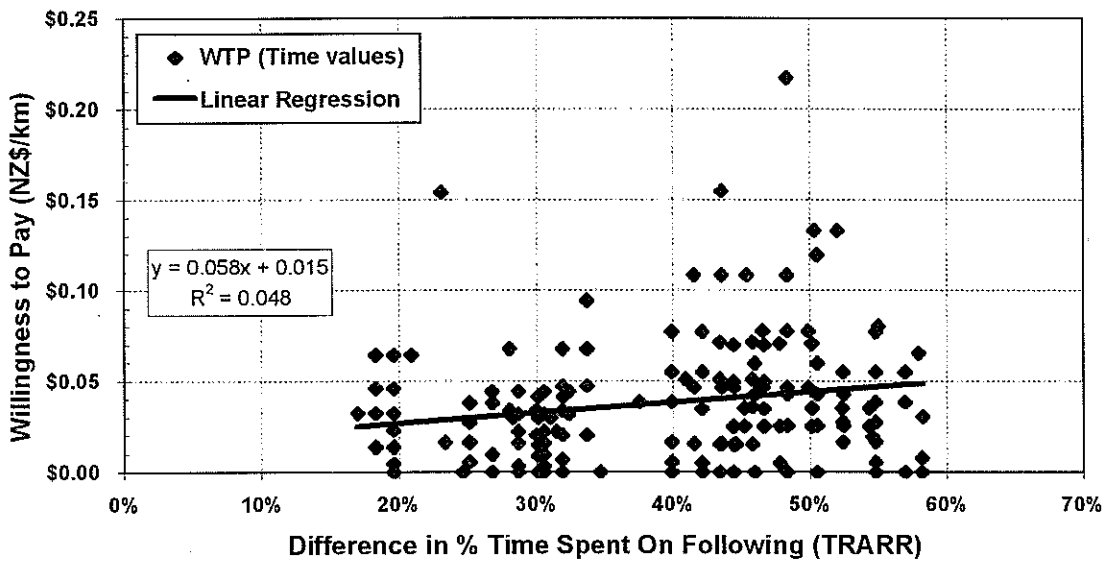
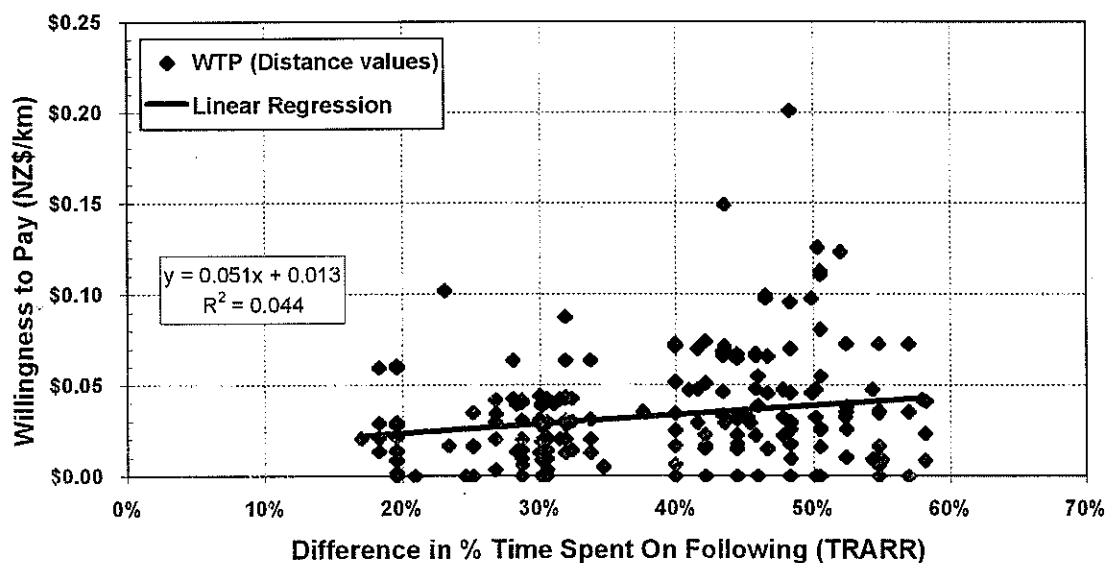


Figure 3.6 WTP (Distance Values) for reduction in PTSF (% time spent following).



Statistically the relationship between WTP and Difference in PTSF is significant at 99% level, though the R^2 values of the best linear fits are very low. The plots in fact bear a similar resemblance to the UPD-based plots in the original analysis.

One of the problems is that there is still quite significant variation in people's WTP. Some people will not accept any WTP value, while other people appear to desire quite a high value. Any possible trend is being masked by this variance. To attempt to resolve this the data was divided into six groups according to the rounded values of differences in PTSF: 20%, 30%, 40%, 50%, 60%.

Figure 3.7 Grouped WTP (Time Values) for reduction in PTSF (% time spent following).

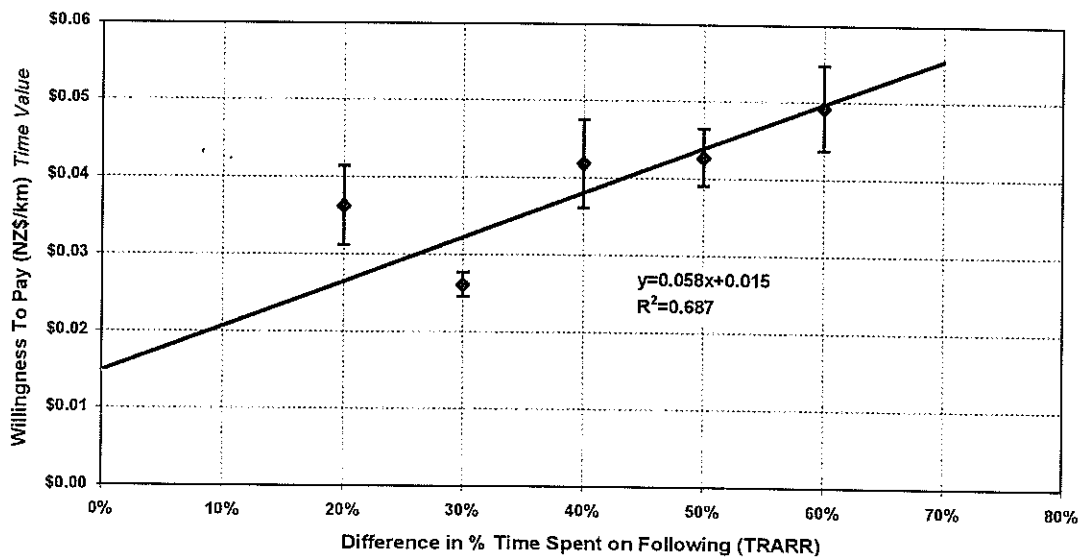
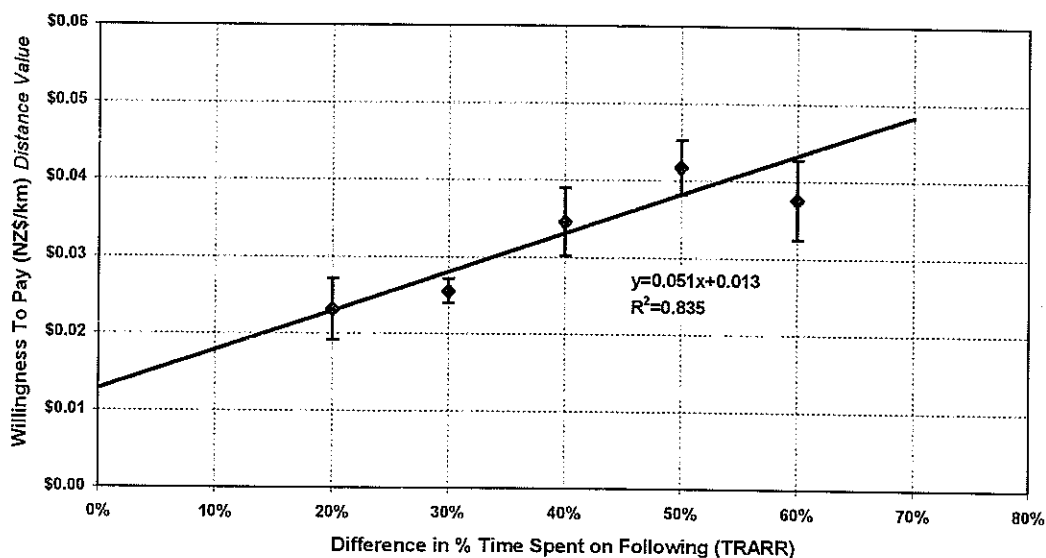


Figure 3.8 Grouped WTP (Distance Values) for reduction in PTSF (% time spent following).



Statistical analysis of each group then allows us to plot the relationship between the difference in PTSF and the averaged WTP for each group (Figures 3.7 and 3.8). The error bars show the standard error for each group (= standard-deviation / $\sqrt{\text{sample-size}}$).

The results show a much clearer trend, although the actual relationships are still the same. By averaging the two time and distance relationships, a reasonable WTP formula to use is:

$$\text{WTP} = 1.4 + 5.4 \times \{\text{Diff-PTSF}\} \quad (\text{cents/km}) \quad (1)$$

It is interesting to note the presence of a constant, certainly large enough to suggest that the regression (assuming it is linear) should not have a zero intercept. This tends to suggest that, like the previous WTP assumed, there is a base value that people will pay just for the presence of passing lanes, irrespective of their effects on travel times and time spent following.

Because of the way that the results were derived for this research, the cost per km can be applied to the length that the change in PTSF affects, rather than just to the length of the passing lane. Generally if a longer analysis length is used, the overall change in PTSF will be less, so the total WTP per vehicle is likely to balance out. One concern with this application in practice is that the typical PTSF reductions will often be less than 10% because of the analysis length chosen, which is considerably less than the actual PTSF reductions used in the derivation. Another concern is that the constant value will mean that longer analysis lengths will produce greater benefits for the same passing lane analysed.

A reasonable interpretation could be to apply the 1.4c/km constant (from Equation 1) to only the passing lane length, while applying the variable part to the entire analysis length. So, for example, if an 800-m passing lane is modelled within a 12-km length and produces a 5% (absolute) reduction in time spent following at a given traffic volume, it will derive:

- a WTP of 1.4c/km over the passing lane length = $1.4 \times 0.8 = 1.1\text{c/veh}$; and
 - a WTP of $5.4 \times 0.05 = 0.27\text{c/km}$ over the analysis length = $0.27 \times 12 = 3.2\text{c/veh}$,
- for a total of 4.3c per vehicle in the direction of the passing lane.

This approach then allows different WTP values to be derived for different traffic volumes, by adjustment of the second WTP calculation.

3.3.3 Determination of PTSF using the Simplified Model

As discussed above, while PTSF can be easily obtained from a TRARR model, it is less clear how it can be obtained from Transfund's simplified model. Although further improvements to this model will be investigated later in this Section 3, a brief consideration of this problem is given here.

The simplified model uses changes in Accrued Passing Demand (APD) to assess the amount of time that vehicles spend waiting to overtake vehicles. Over the analysis length, this APD is summed to produce an Overall Passing Demand (OPD) which is then applied to the Average Time Lost by following vehicles, to determine travel time costs. It is assumed that the APD is generally proportional to the level of bunching and the traffic volume, so that:

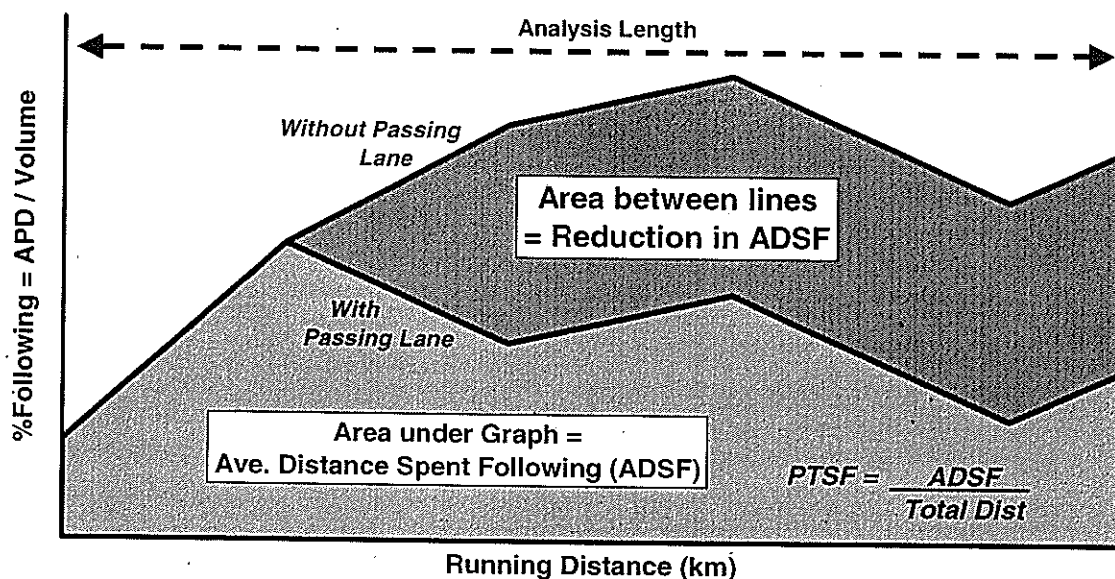
$$APD = \{ \%Bunching \} \times \{ One-Way-Volume \} \text{ (o'takings/hr)} \quad (2)$$

By dividing APD by the traffic volume, the change in bunching (or following) can be modelled down the road instead. The overall area under a plot of this will represent the Average Distance Spent Following (ADSF), i.e.

$$ADSF = \int_0^{AnalysisLength} \%Following \quad (3)$$

The PTSF can then be determined by dividing ADSF by the total distance, i.e. it represents the average %Following along the analysis length. By comparing the change in ADSF with and without the passing lane in place, the reduction in PTSF can be calculated. Figure 3.9 shows this graphically.

Figure 3.9 Derivation of PTSF from APD analysis.



Although the analysis is based on (easier to survey) spot-bunching measures (i.e. %Vehicles Following) rather than on PTSF measures, PTSF approximates bunching fairly well. Harwood et al. (1999) found that using a 3-second headway⁶ in the field when measuring bunching produces the best estimation of PTSF.

⁶ Headway – time gap between the front of successive vehicles.

3.4 Improvements to the Simplified Model

The existing PEM simplified passing lane procedures aim to provide a simple but realistic means of assessing the effects of providing a passing opportunity, and passing lanes in particular. Although comparison of benefits from TRARR simulations indicated that the simplified procedure produced similar results, a number of conceptual issues still remain to be addressed. Improvement of these would help improve the validity of the model in the eyes of both the users (roading practitioners) and their clients.

3.4.1 Issues Requiring Investigation

Some of the issues that have arisen include:

- It is not fully understood how the measure of Accrued Passing Demand (APD), used to assess travel-time savings lost, relates to more easily measured values such as bunching levels. This is particularly important to allow verification of the simplified model using field measures. Bunching surveys can currently be used to specify the initial APD, but this relationship needs to be confirmed in the field.
- The simplified procedures do not adequately reflect the diminishing effect of benefits downstream of a passing lane. TRARR analyses for example will show that bunching levels for both the existing road and proposed passing lane option will gradually converge again downstream, the length required being dependent on volumes and terrain. At present the simplified model produces APD levels (a proxy for bunching) for each of the options that generally run in parallel. To counter this, the downstream benefits are currently limited to just 5 km beyond the end of the passing lane, but there may be sites where still more benefits accrue further away.
- The procedures currently use a theoretical constant value when determining the available supply of passing opportunities. How this value relates to actual maximum passing levels or whether it should vary in different situations is not clear.
- The simplified procedures have not been tested on longer strategic studies involving multiple passing lanes (hence, they are currently only allowed for use on single passing lane projects). This issue relates back to the difficulty in being able to use field measures along the road section to validate the model.
- The PEM simplified method is still somewhat cumbersome to apply, partly because of the field data requirements. In particular, the need for various speed values along the highway for various vehicle streams is rather onerous. A balance needs to be struck here between providing realistic inputs to the simplified model and in preserving the simplified nature of the analysis.

The simplified procedures may also not be appropriate for other passing opportunities, such as slow vehicle bays. This is examined further in Section 4 of this report.

To investigate these issues further, a number of approaches were taken:

- Recent overseas and local research was reviewed to consider theoretical refinements to the simplified model.

- Field data from selected routes were collected and analysed. This was compared with simplified models of the equivalent sections.
- Various TRARR test runs of extreme scenarios were used to confirm field data findings (or to test out situations not easily replicated in the field).

The findings are discussed in Section 3.3.2.

3.4.2 Accrued Passing Demand and Bunching

As discussed in Section 3.2.3, the Accrued Passing Demand (APD) is generally considered to be proportional to the level of bunching and the traffic volume, so that:

$$APD = \{ \%Bunching \} \times \{ One-Way-Volume \} \text{ (o'takings/hr)} \quad (4)$$

This implies that every vehicle following in a queue has an immediate desire to pass only one vehicle (namely that at the head of the queue). It was recognised by Koorey et al. (1999) however that this would under-estimate APD, where queued vehicles also wish to overtake other vehicles ahead of them in the queue. For example, an existing queue of vehicles may have encountered an even slower vehicle to create a larger queue. On relatively low-volume highways with minimal bunching, the effect should not be great, although more work was needed to confirm how much of a difference it made.

For a bunch observed at any point, it is impossible to determine the actual passing demands within the queue. The only known fact is that all queued vehicles must have faster desired speeds than the lead vehicle. For every other interaction between vehicles we can only assume a 50/50 chance that the trailing vehicle wishes to pass the vehicle in front.

Consider for example a bunch of three vehicles. Both trailing vehicles will have higher desired speeds than the lead vehicle, so there is a minimum passing demand of two. However the third vehicle may also wish to pass the second vehicle, resulting in a maximum passing demand of three. Statistically the average passing demand is 2.5. This exercise can be continued for longer queue lengths to determine a pattern, as shown in Table 3.4.

Table 3.4 Average passing demand within different bunch sizes (B).

Bunch Size	Minimum Passing Demand	Maximum Passing Demand	Average Passing Demand	Queued Vehicles	Ratio: Ave PD / Queued Veh
1	<i>Single Vehicle Only</i>		0.0	0	-
2	1	1	1.0	1	1.00
3	2	3	2.5	2	1.25
4	3	6	4.5	3	1.50
5	4	10	7.0	4	1.75
<i>B</i>	<i>B-1</i>	<i>B(B-1)/2</i>	$[B(B+1)-2]/4$	<i>B-1</i>	$(B+2)/4$

By knowing the distribution of bunch sizes in a given traffic stream, it would be possible to determine the relative difference between the estimated APD (calculated by %Bunching) and the true APD.

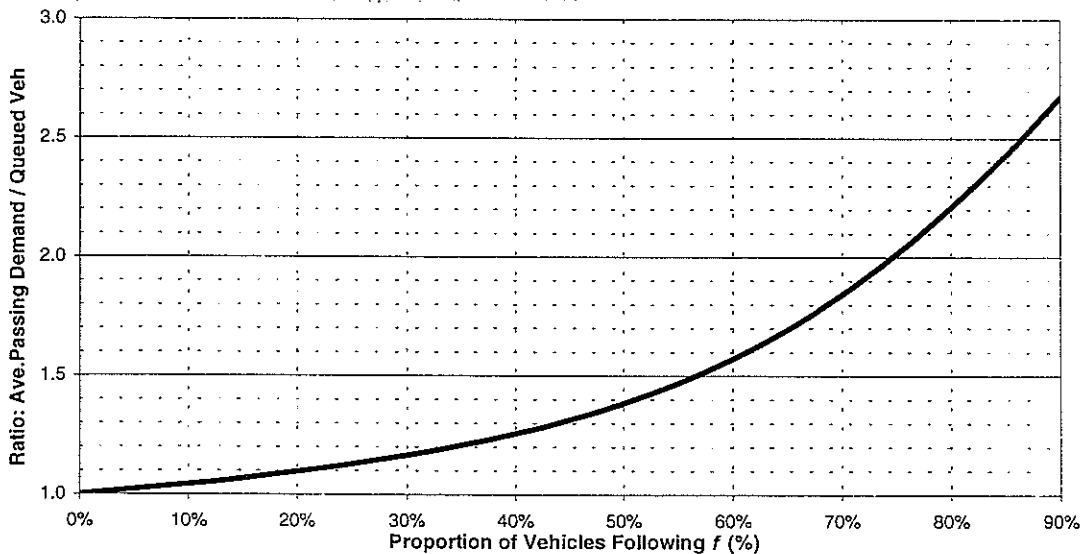
The Borel-Tanner Distribution (BTD) provides a reasonable model for describing the distribution of bunch (or platoon) sizes in traffic on two-lane, two-way rural roads. The probability of a bunch size B is given by

$$P(B) = [Bf \exp(-f)]^{B-1} \frac{\exp(-f)}{B!} \quad (5)$$

and for $B = 1, 2, 3, \dots$, where f is the proportion of vehicles following. This is also related to the mean bunch size $m_B = 1/(1-f)$, or alternatively, $f = (m_B - 1)/m_B$.

Given an observed proportion of vehicles following, the BTD can provide a frequency distribution of bunch sizes. By applying this distribution to the ratios derived in Table 3.4, the average ratio of APD per Queued Vehicle, $R(f)$, can be determined. Figure 3.10 shows the results when $R(f)$ is calculated over a range of vehicle-following proportions.

Figure 3.10 Ratio $R(f)$ of APD (Average Passing Demand) per queued vehicle.



$R(f)$ cannot be described by an exact function, but the function $(1 + 2.6052 f^3 - 0.9103 f^2 + 0.5692 f)$ fits the curve with an R^2 of 0.999. In this way, the true APD from observed bunching can be established:

$$\text{True APD} = f \times \{\text{OneWayVolume}\} \times R(f) \quad (\text{overtakings/hr}) \quad (6)$$

For example, if a highway with 300 veh/hr (one-way) was observed to have 40% bunching, then the true APD would be $40\% \times 300 \times 1.26$ (from chart) = 151 overtakings/km/hr. By applying the above correction factor, it should be possible to better relate observed bunching proportions with calculated APDs.

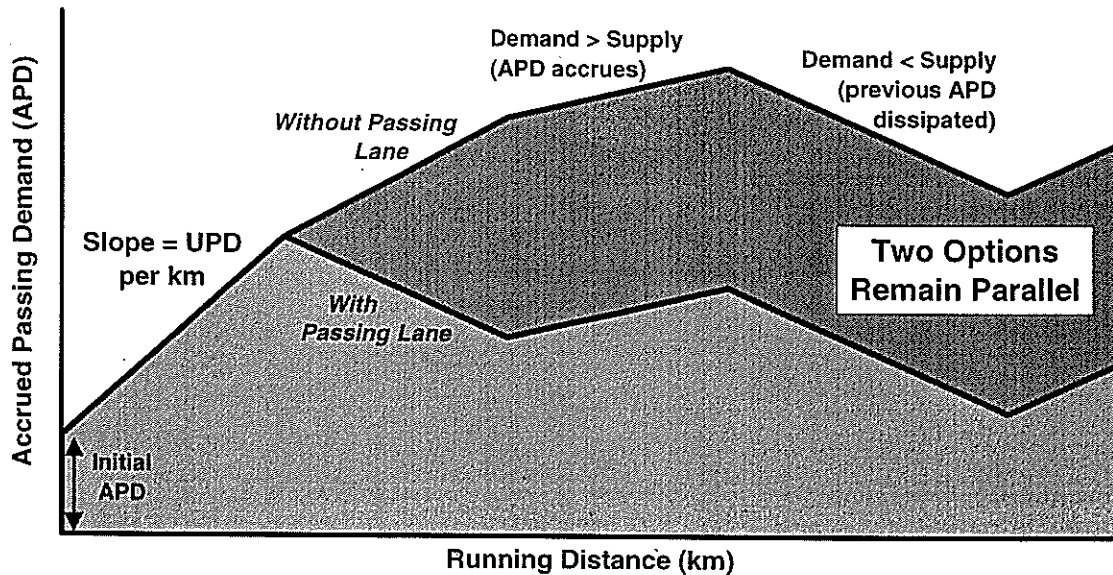
3.4.3 Variations and Constraints on Passing Demand

The existing simplified procedure combines relationships for passing supply and demand together to determine the Unsatisfied Passing Demand (UPD) in each road segment. This is then applied to the segment length to determine the relative increase or decrease in Accrued Passing Demand (APD). For a given road segment, UPD

currently does not vary, irrespective of the prior passing demand (with the exception that APD cannot fall below zero). When comparing passing lane options, this therefore generally leads to parallel changes in APD following a passing lane, with the two options never meeting.

Figure 3.11 illustrates this scenario. Given that passing lane effects generally diminish downstream over a fixed length, this result is not producing a realistic model of the situation.

Figure 3.11 Typical derivation of APD (Accrued Passing Demand).



Another potential problem is that a road with poor existing passing opportunities and high traffic volumes can suffer from constantly high UPDs, with the net result that the APD continues to increase unconstrained. Notwithstanding the previous discussion on average passing demands per queued vehicle, there is clearly a limit to how much APD is present at any given point along the road, particularly when the proportion following is not observed to be exceptionally high. This is because only a finite number of vehicles are present.

Section 3.3.4 will consider the “passing supply” side of the equation, particularly with regard to maximum passing rates. Let us consider for now, how the “passing demand” is derived. The number of “catch-ups” determines demand. The Passing Demand, D_{A-B} , (i.e. the frequency with which vehicles in stream A catch up to vehicles in stream B) is determined by:

$$D_{A-B} = Z \times K_A \times K_B \times \sigma_A \quad (\text{catch-ups/km/hr}) \quad (7)$$

where:

- Z = a constant, based on relative differences in speed between the two streams A, B
- K_A = Traffic Density = $\{HourlyFlow\}_A / \{MeanSpeed\}_A$ (veh/km) for streams A, B
- σ_A = Standard Deviation of traffic stream A (km/hr)

Often the two traffic streams analysed are “cars” catching up to “trucks”. This formula can also be simplified to determine catch-ups within a traffic stream, e.g. cars catching up to slower cars.

The original basis for these calculations was for low traffic flow cases where single vehicles caught up to other single vehicles and were likely to be able to overtake relatively soon after catching up. A maximum one-way flow of 150 veh/hr, or a mean bunch size of less than two, was recommended for using this model in order for the results to be reasonably valid. Given that, in many cases, the flows modelled are likely to be somewhat greater, the true demand needs to be considered in these cases.

When determining catch-up rates when some vehicles remain bunched, remember that only the lead vehicles or free vehicles will be dictating vehicle interactions. The other vehicles already bunched are not affecting the additional catch-ups generated, and could be excluded from the passing demand calculations, if we concentrate only on the minimum passing demand and do not include within-queue interactions.

As the proportion of bunched vehicles increases, the proportion of vehicles generating catch-ups should decrease. For a given % following, f , there are $(1-f) \times \{Volume\}$ vehicles free to interact. The calculated passing demand is hence $(1-f) \times D_{A-B}$. If we assume that APD is proportional to bunching and volume, then this calculation could be re-expressed as:

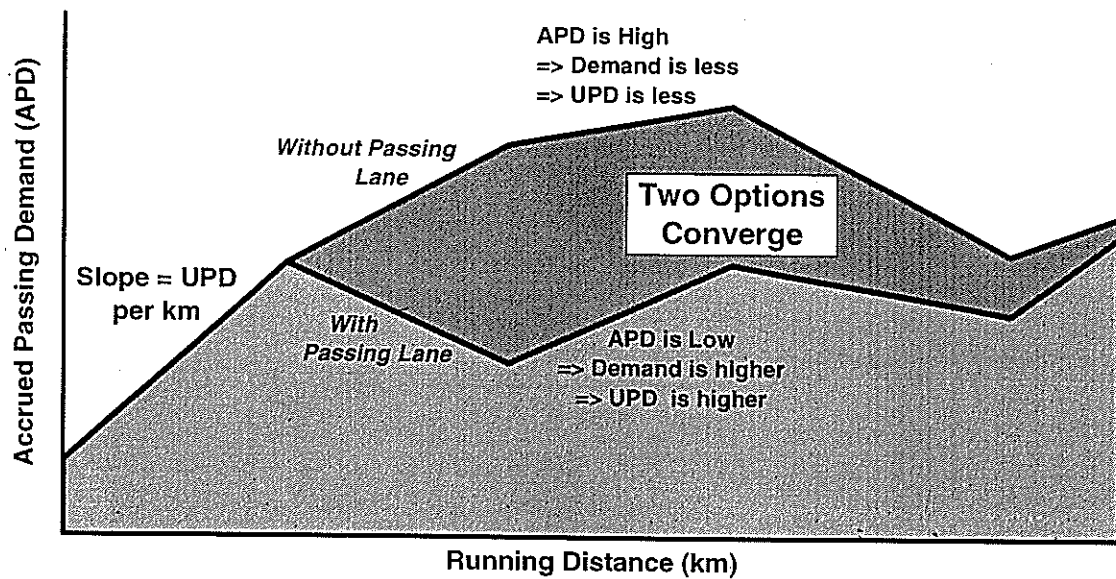
$$\{TrueDemand\}_{A-B} = (1-APD/\{OneWayVolume\}) \times D_{A-B} \quad (8)$$

The net effect of this approach is that, at relatively high levels of APD, the passing demand generated is much lower than at low levels of APD for the same road segment. This has the effect of lowering the UPD and subsequently not increasing the APD by as much (or decreasing it by more). A passing lane option that will substantially lower the APD is likely to cause a quicker return downstream to a higher APD than that for the same road segment without the passing lane.

This results in the two options converging again at some point downstream, and Figure 3.12 illustrates this effect.

The above approach will also help to minimise the rate of climb in APD as, when APD is approaching the actual volume, UPD values will be scaled to near zero. Because of the non-continuous nature of simplified passing lane procedures, where road segments can often be a few kilometres in length, an additional cap will be necessary to prevent APD over-runs. Given that it is highly unlikely to see bunching greater than 90%, a reasonable approach is to cap APD to a maximum of $0.9 \times \{OneWayVolume\}$. Like the zero APD restriction, this limit will apply until a negative UPD is found downstream. It may also be sensible to limit road segment lengths to no more than 2-3 km to provide some additional precision in the results.

Figure 3.12 APD (Accrued Passing Demand) adjusted for bunching.



3.4.4 Maximum Passing Rates

In Transfund’s simplified PEM procedures, it is suggested that the maximum passes on conventional two-way highways are about 108 vehicles per kilometre per hour. The existing procedure calculates passing supply, S (overtakings/km per hour) as:

$$S = PASD \times PAG \times 108 \quad (9)$$

where:

PASD = Proportion of Available Sight Distance (= 1 for passing lanes)

PAG = Proportion of Available Gaps in opposing traffic (= 1 for passing lanes)

Koorey et al. (1999) assumed that if a vehicle required on average 30 seconds to safely overtake, then it could theoretically overtake $3600/30=120$ vehicles every hour. In reality, there might be some inefficiency of, say, 10% bringing this figure down to 108 overtakings/hr. This value is fixed and does not reflect the influence of traffic density. As the value has not been confirmed in the field, further investigation is warranted.

Harwood & St John (1984) revealed the strong relationship between passing rates (at sites treated using passing lanes) and flow rates, represented by the regression model:

$$PR = 13.0 + 0.223 FLOW: \text{ for } 50 \text{ veh/hr} \leq FLOW \leq 400 \text{ veh/hr} \quad (10)$$

where:

PR = Passing rate (completed passes per hour per mile) in treated direction

$FLOW$ = Flow rate (veh/hr) in treated direction.

This model explains 47% of variance in the dependent variable (i.e. $R^2 = 0.47$).

An improved regression model for predicting the passing rate in the treated direction was obtained by adding two additional independent variables – passing lane length and upstream percent of vehicles platooned – to the model. The revised model for passing rate in the treated direction is:

$$PR = 0.127 FLOW - 9.64 LEN + 1.35 UPL \quad (11)$$

where:

LEN = Passing lane length (miles)

UPL = % of traffic platooned at upstream end of passing lane

The model explains 83% of the variance in the dependent variable ($R^2 = 0.83$). It shows that the passing rate increases with increasing flow rate and with increasing upstream percent of vehicles platooned. The model also shows that the passing rate decreases with increasing passing lane length. This finding confirms that the passing rate is highest near the beginning of a passing lane and decreases to a lower, steady state level at some distance into the lane.

Fong & Rooney (1997) studied traffic data for five passing lanes. The average distance of the first completed pass by vehicles in groups of three or more vehicles was 309m with a standard deviation of 32m between sites. The average 85th percentile distance of the first completed pass was 495m with a standard deviation of 29m. They suggested that this 85th percentile data be used for planning minimum distances for proposed passing lanes along routes, where there is not much traffic and where typical speeds are about 55mph (88 km/hr) or faster.

This seemed to tie in with research from May (1991), who recommended that passing lanes on the order of 0.25 to 0.75 miles (0.4-1.2 km) appeared to be the most effective. May also suggested that spacing between such passing lanes in the order of 2 to 5 miles (3.2-8.0 km) appeared appropriate, depending on downstream roadway and traffic conditions.

Overtaking behaviour in New Zealand may be somewhat different to that found in the above research. Therefore local field surveys were carried out to examine this behaviour and they are discussed in Section 3.4.

3.4.5 Other Potential Changes

A number of minor suggestions for improvement to the simplified passing lane procedures have been identified, either from use of the existing procedures or from other research findings. These are discussed below.

- BCHF (2000) developed an alternative procedure for assessing passing lanes on a strategic basis, using a similar method to the existing simplified procedure but with some changes. One of the changes deals with the way that overtaking gaps are identified. At the moment there are two controls on overtaking ability, the Proportion of Available Gaps (PAG) in the opposing traffic stream and the Proportion of highway with Adequate Sight Distance (PASD). The former is based on requiring 30 sec gaps, while the latter only looks at sections of highway with sight distance >450m. Both measures are fairly conservative, particularly in situations where the speed differential between interacting vehicles is fairly high (e.g. car overtaking a truck on a steep grade).

BCHF devised an alternative formulation for the “critical gap”, *CGAP*, based on the speed differential, ΔV , between the two interacting traffic streams:

$$CGAP = 5 + 8.36 \times 10^{-8} \times (100 - \Delta V)^{4.337} \quad (\text{secs}) \quad (12)$$

This equation provides a typical range of time gaps between 10 and 30 seconds, depending on the speed differential, with a maximum gap of 44 seconds where differential is normally evident. This gap can then be used to determine both the minimum sight distance length required at a point as well as the time gap required in the opposing traffic stream. Although BCHF could calculate both of these continuously by means of a software program, for general evaluation use it might be impractical to adjust the minimum sight distance length for every road segment in the current procedures. However, with knowledge of the car and truck speed distributions on each segment already available, the time gap for opposing traffic could be incorporated easily. PAG could then be calculated by:

$$\text{PAG} = e^{-CGAP/3600 \times \{OneWayVolume\}} \quad (13)$$

- The existing procedures require two assessments of speed. First the mean free speeds of cars and trucks are required as well as their standard deviations; these are used to determine the passing demand interaction. Second the mean free and following speeds of all vehicles are required to identify the likely time savings per overtaking. Although all this data is available by analysis of speeds and headways from an automated traffic classifier survey, in many cases such a survey is not considered feasible. Free speeds can often be determined by simple spot surveys or by derivation of vehicle mechanics on grades. Following speeds however are harder to identify without headway information.

Given that free speed distributions are more easily obtained, it seems pragmatic to make use of these in determining the likely increases in speeds. For interactions between traffic streams, the difference in mean free speeds can be taken as the average likely value. For interactions within traffic streams, the difference between the 85th percentile speed and the mean speed provides a typical value (statistically, the mean difference between any two vehicles is slightly larger). Assuming a normal distribution, this difference is conveniently approximated by the standard deviation of free speeds. The mean time saving overall can then be determined by weighting the differences calculated above the relative proportion of demand in each case.

3.5 Local Survey of Maximum Passing Rates

A local survey was conducted to determine maximum overtaking rates for New Zealand. The survey was conducted at a northbound passing lane located at SH1 Otaihanga (RP 931/7.2-6.2), south of Waikanae, North Island, in October 2000. The passing lane is 1000m long (excluding tapers) with a high volume of traffic (AADT ~ 21,000) and on a high-speed alignment. It is expected that many vehicles would use the passing lane to overtake. Part of the site also has a passing lane in the opposing direction, however this was not expected to affect the operation of the surveyed passing lane.

3.5.1 Method

A series of traffic counters were set up on site for approximately one week. The proposed survey method was to collect bunching and count data beyond each end of the passing lane, as well as similar data at regular intervals within the passing lane.

Initially it was hoped to be able to collect count data from both northbound lanes by means of “staggered” tubes, half of which covered both lanes and half which covered only the slow lane. However practical trials showed these to be difficult to maintain, with the tubes coming loose and being damaged far too frequently. The alternative was to just monitor the slow lane; subtraction from the counts at either end allowed the use of the passing lane to be determined. Because some overtaking manoeuvres would finish early or start late, differences between the three passing lane counts could be used to infer the probable likely number of passes made. Figure 3.13 shows the final layout used.

Figure 3.13 Layout of Otaihanga passing lane survey.

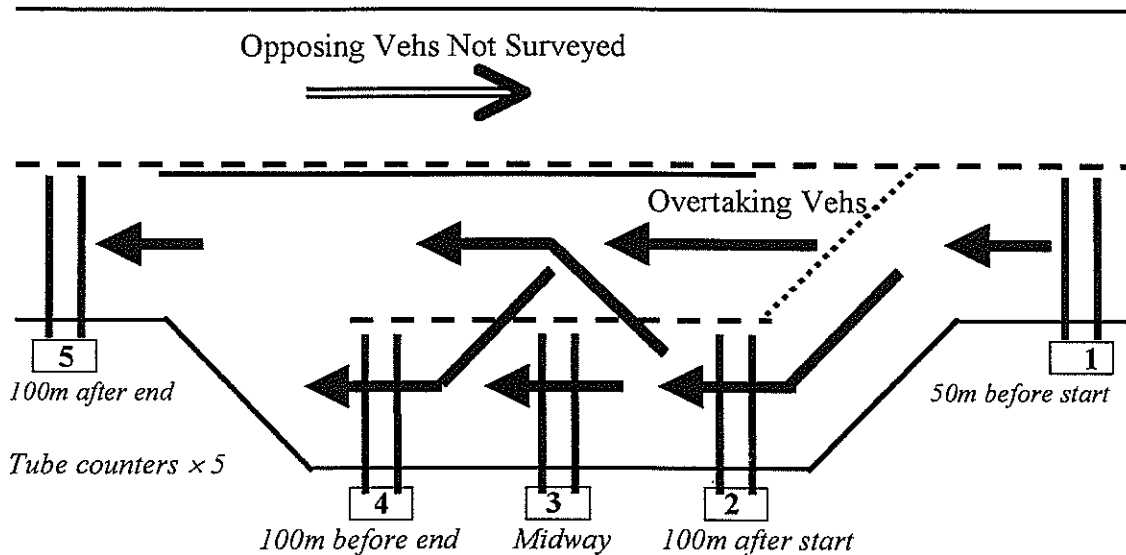
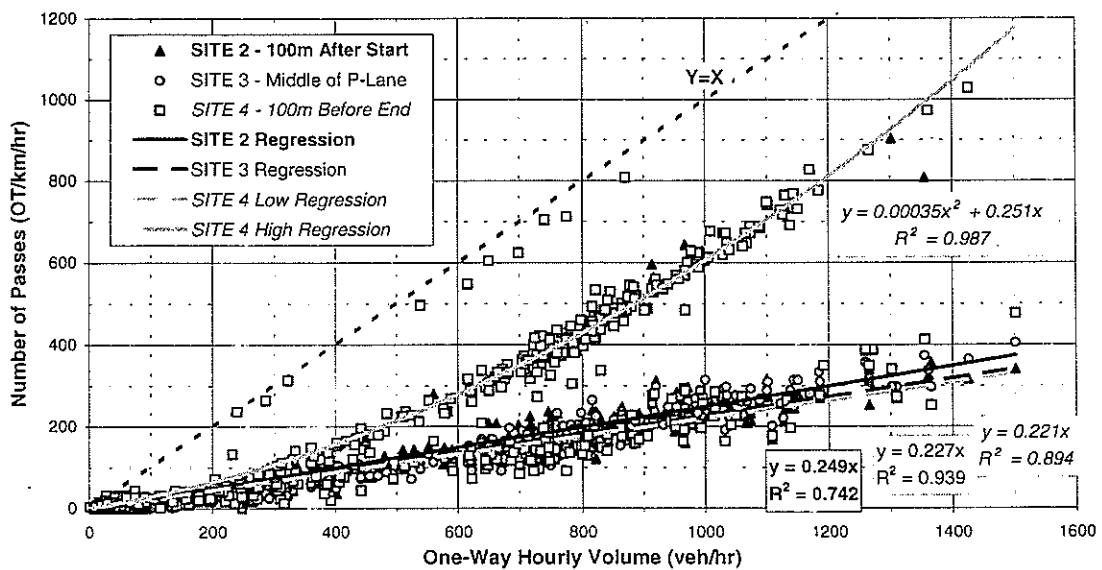


Figure 3.14 Number of passes observed on Otaihanga passing lane.



The collected survey data was inspected and spurious time periods (mainly due to faulty tubes) were removed from the analysis. 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.

3.5.2 Results

The plot shows some interesting characteristics. Firstly there is a clear linear trend between the number of passes undertaken and the volumes, with approximately 22%-24% of traffic using the passing lane at any given flow. The relationships are in fact slightly curved upwards, but the linear assumption provides a sufficient fit. At Site 4 however there is a distinctive dichotomy between those periods that follow this pattern and those periods where a much higher proportion of vehicles are in the passing lane. For these higher passing numbers a polynomial relationship as shown gives a much better fit than a linear one. Note too that there are even a few periods where the flow in the passing lane almost matches the overall flow (dashed line) possibly because of some obstruction in the slow lane.

The Site 4 results suggest that slower vehicles are moving over and merging with the overtaking traffic in advance of the final merge taper. A few points at Site 2 also show this higher trend, suggesting some late movers to the slow lane, although the effect is not as noticeable. Placement of these two sites a further 100m in towards the centre of the passing lane may have removed some of these effects.

Discussion with the survey contractor revealed a problem at Site 4 with the vehicles moving over hitting the ends of the tubes and causing them to fail. As a result, the tubes were being re-attached daily and appeared to be operating successfully for about 12 hours each time before failing. Based on this information, the higher regression for Site 4 is likely to be more representative of driver behaviour at this location.

In trying to assess the number of overtakings occurring overall at this site, it is important to remember that overtaking vehicles may pass more than one counter while in the passing lane. For example, a vehicle might immediately start overtaking at the beginning of the passing lane and move over after 700m, in which case it would be recorded as being in the passing lane at Sites 2 and 3 but not 4. The fact that the rates at Sites 2 and 3 are very similar suggests that this is quite common and that both sites are recording the same overtaking vehicles. If many additional overtakings were commencing later in the passing lane, one would expect to see a higher overtaking rate at Site 3, but this does not appear to be the case. It may be that most overtaking vehicles are remaining in the passing lane throughout its length. Certainly at higher volumes, where lane changes are more difficult, this is understandable.

Although the presence of a vehicle in the passing lane does not necessarily constitute a successful overtaking, conversely, a vehicle in the passing lane may be able to overtake a number of vehicles depending on its relative speed differential.

3. Field Calibration of Simplified Procedures

The previous work by Fong & Rooney (1997) suggested that most vehicles could complete an overtaking manoeuvre in less than 500m; therefore it should be possible to achieve two passes per overtaking vehicle at Otaihanga over its 1000m length. In fact the rate is likely to be higher because additional acceleration is usually not needed for further “flying” overtakings (i.e. where the overtaking vehicle is already at a higher speed than the overtaken vehicle). Taking the observed passing lane use rate of approximately 22%-24%, this suggests an overtaking rate per kilometre of about half the observed volume, i.e.

$$PR \sim 0.5 FLOW \quad (14)$$

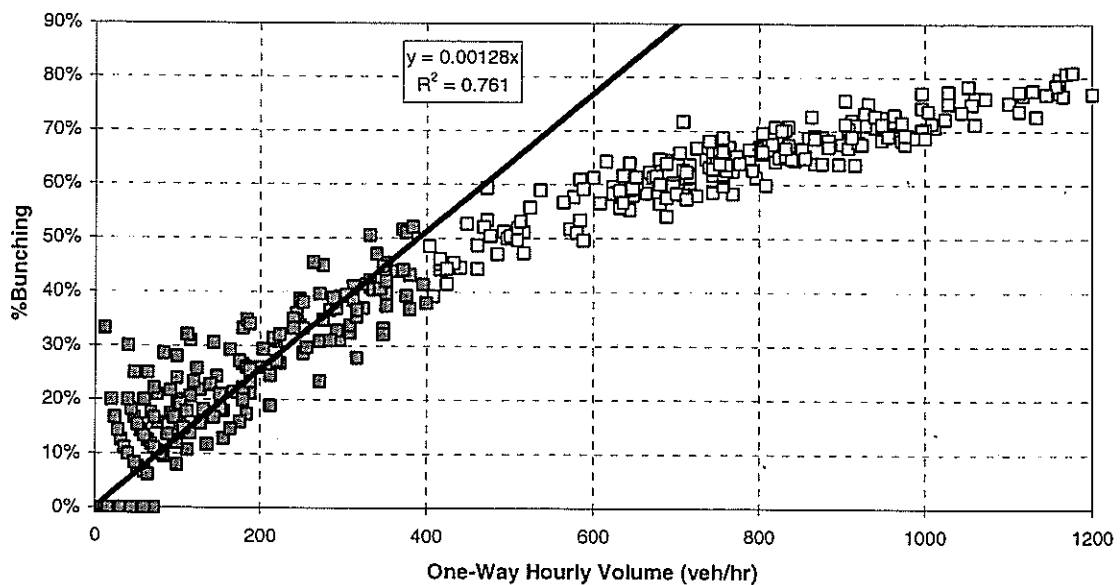
For example, at 600 veh/hr (one-way), a maximum rate of ~300 overtakings/km/hr is likely.

It is interesting to compare this finding with the passing rate derived by Harwood & St John (1985). When adjusted for metric units, their equation (in passes per km) becomes:

$$PR(km) = 0.0794 FLOW - 3.77 LEN(km) + 0.844 UPL \quad (15)$$

Bunching data from Site 1 was compared with hourly volumes to determine a relationship for UPL. Figure 3.15 plots the results, showing the typical negative exponential shape. In keeping with the original analysis, only readings below 400veh/hr (dark squares) have been used to derive a simple linear relationship.

Figure 3.15 Bunching levels on highway before Otaihanga passing lane.



The UPL relationship derived is $\%UPL = 0.128 \times FLOW$. Incorporating this with the previous equation, and allowing for the 1000m passing lane length, produces:

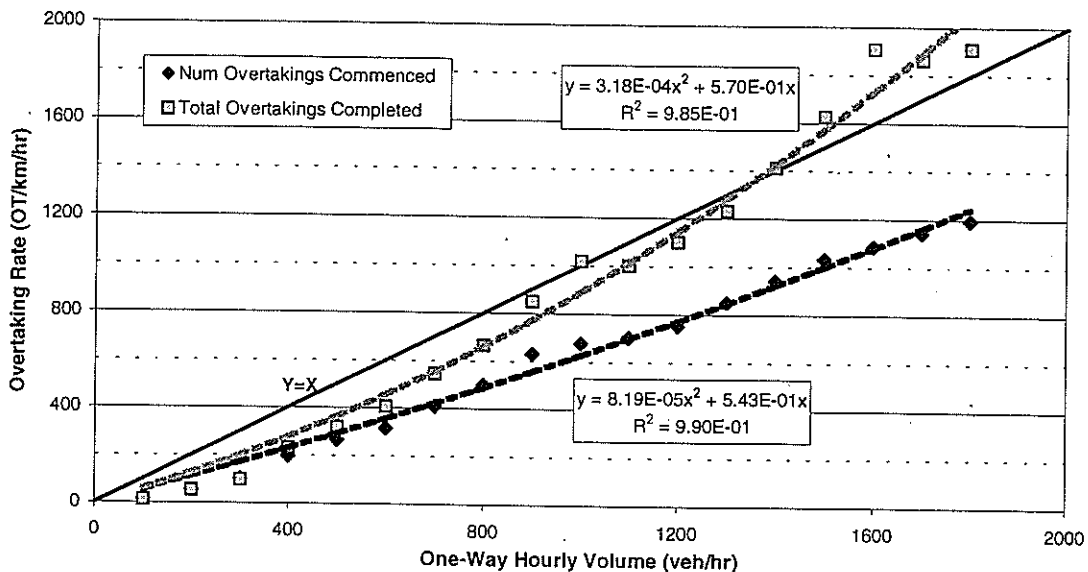
$$PR(km) = 0.187 FLOW - 3.77 \quad (16)$$

While this is similar to the number of passes observed throughout the Otaihanga passing lane at the survey points, it is much less than the estimated actual overtaking rates discussed above.

3.5.3 Comparison with TRARR Model

For comparison, a TRARR model was built to simulate the Otaihanga passing lane and surrounding highway between Waikanae and Paraparaumu. The model was calibrated so that bunching levels prior to the passing lane matched those found in the field. Traffic was modelled at different flow levels up to 1800 veh/hr (one-way) and observation points were placed within the passing lane. Two different measures were used from the TRARR results, scaled to take account of the simulation time. First the number of overtakings that commenced within the passing lane was observed (this will include those that are completed after the passing lane end). Second the total number of overtakings completed within the passing lane was observed (this will include those that began before the start of the passing lane). By the nature of their recording methodology, both measures may record overtakings not fully within the passing lane. Figure 3.16 shows how the two measures vary with volume.

Figure 3.16 Overtakings from Otaihanga passing lane TRARR model.

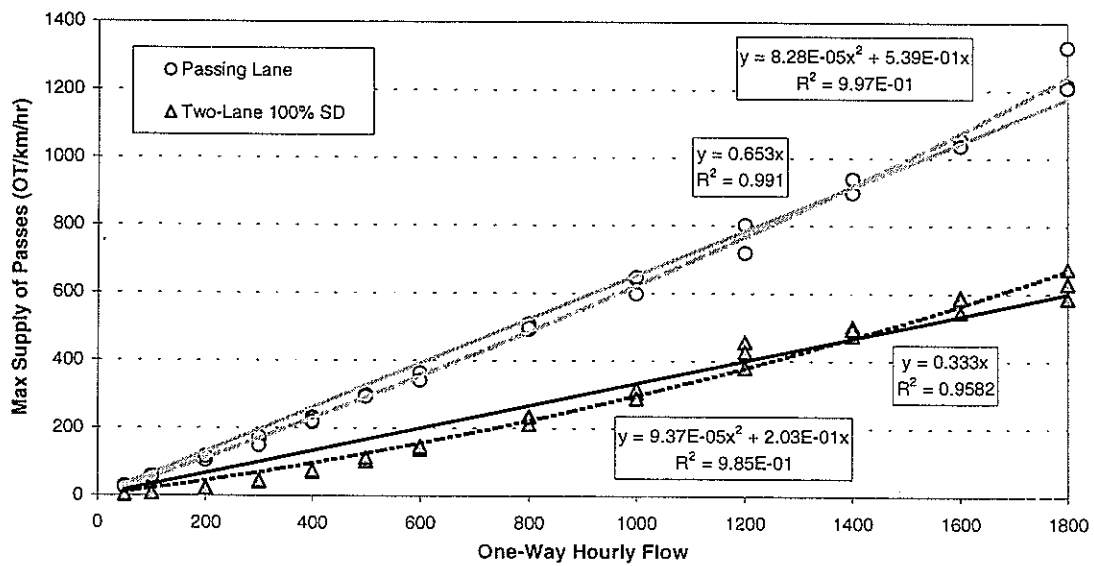


Both measures display somewhat curved relationships, particularly that for completed overtakings. Evidently a great proportion of overtakings begin before the passing lane, presumably when there is a clear opposing gap before reaching the passing lane. However inspection of the data revealed very few overtakings beginning within the 500m before the start of the passing lane, especially at high volumes, suggesting some imprecision about the way that TRARR records overtaking positions. Looking at the overtakings that commenced within the passing lane, over 80% began within the first 200m, irrespective of volume.

A site like Otaihanga may not provide the optimum opportunity for vehicles to overtake. The above passing lane also does not provide information about maximum passing rates on two-lane highways without passing lanes. Therefore another TRARR model was built to simulate a segment of straight and flat road with maximum sight distances. This time two alternatives were considered, one with just a

continuous two-lane highway, and one with a continuous passing lane (with no passing allowed before it). The volume of the opposing traffic was set to zero to maximise passing opportunities. In such cases all the following vehicles are technically free to pass and the passing rate is expected to reach its maximum level. At each volume level, a number of different random seeds were run to obtain a distribution of passing rates. Invariably within passing lanes, the first kilometre had the highest passing rate (based on the number of overtakings commenced) and so these figures were used. For the two-lane highway the 500m section with the highest overtaking rate was used. The outputs from the TRARR runs reveal the relationship between maximum passes and traffic volume as shown in Figure 3.17.

Figure 3.17 Maximum passing rates modelled by TRARR.



Clearly, the plots show that the available traffic volume has to be factored in. It is also very evident that, in passing lane situations, the rate is considerably higher. The similarity with the estimated passing lane rate from the Otaihangā field survey (using the polynomial regression) also validates the finding. The 108 overtakings/km/hr previously estimated is clearly conservative when traffic volumes are not low. Although the linear regressions appear to provide a very adequate fit to the relationship overall, given that they over-estimate passing at the low volume (where most analyses will be), the polynomial relationships should be used.

3.5.4 Changes to Simplified Procedures

The revised suggested calculations for passing supply (S) are now:

$$S = PASD \times PAG \times [0.20V + 0.00009V^2] \quad \text{(two-lane sections)} \quad (17)$$

or:

$$S = 0.653 \times [0.54V + 0.00008V^2] \quad \text{(passing lanes)} \quad (18)$$

where:

V = the one-way hourly volume.

The net result of these changes is to reduce the supply of passing available on two-lane roads in proportion with the traffic, and to increase the supply of passing available on passing lanes.

The above passing lane equation produces the result that APD is likely to dramatically drop, often to zero, within a passing lane. However the TRARR findings indicate that passing rates drop off dramatically soon after the start of the passing lane, and bunching generally does not fall to zero or even to very low values. This suggests that these maximum rates are only achieved where there is an evident need, i.e. when bunching is high. Once this high bunching has been reduced, or if the passing lane was in an area with low bunching anyway, then a lower level of passing will occur. It may be that, like the determination of passing demand, passing supply needs to be scaled relative to APD. This time however the relationship would need to be reversed, i.e. make supply proportional to $APD / \{OneWayVolume\}$.

3.6 Comparison of Improved Simplified Procedure with Field Data

To try to validate the above changes, a modified simplified model was set up (taking into account the changes suggested above) for an 18km section of highway containing three northbound passing lanes. The road, part of SH3 between Wanganui and Waverley, had been the subject of a recent passing lane study. As a result, field data and a TRARR model was available to provide comparisons. The section studied here is only a part of the entire length studied but, in having three passing lanes in close proximity, it provides an opportunity to test the robustness of the simplified procedure over more complex situations. Note that only one direction (northbound) has been analysed here.

Figure 3.18 shows the bunching data collected previously along the route and the %Following outputs from the equivalent TRARR model. Both sets of data are for traffic flows of 150 veh/hr, which is a typical daytime flow on this section; a very similar pattern was also produced at 50 veh/hr. Looking at the location of the passing lanes, their effects can be clearly seen. Note that a perfect calibration with the spot bunching data has not been achieved. This suggests that either local “disturbances” (e.g. traffic from nearby side roads) are causing artificially high bunching at some sites, or that TRARR is not being restrictive enough in modelling limited passing opportunities away from the passing lanes. Because of the more limited field data available, comparison with the simplified model has focused on its match with the TRARR model.

A simplified model of the route was built using ten homogeneous segments. The free speed data was based on the TRARR outputs at low volume, to provide a valid comparison. Figure 3.19 shows the resulting APD plot at 150 veh/hr. From the previous discussion (in Section 3.3.1) about APD and bunching, reasonable inferences about the bunching exhibited can be made, e.g. an APD of 30 is equivalent to 20% (=30/150) bunching.

3. *Field Calibration of Simplified Procedures*

Although the APD plot produces a similar shape to the bunching plot shown in Figure 3.18, it is clearly displaying greater extremes. APD values equivalent to more than 40% bunching are calculated prior to the first passing lane, while the effects of the first two passing lanes are to reduce bunching by about 20% each.

Compared with the field bunching data, APD appears to over-estimate, as opposed to TRARR's apparent under-estimation. The same pattern is evident when comparing the APD plot for 50 veh/hr.

Figure 3.18 TRARR and field bunching data for SH3 north of Wanganui.

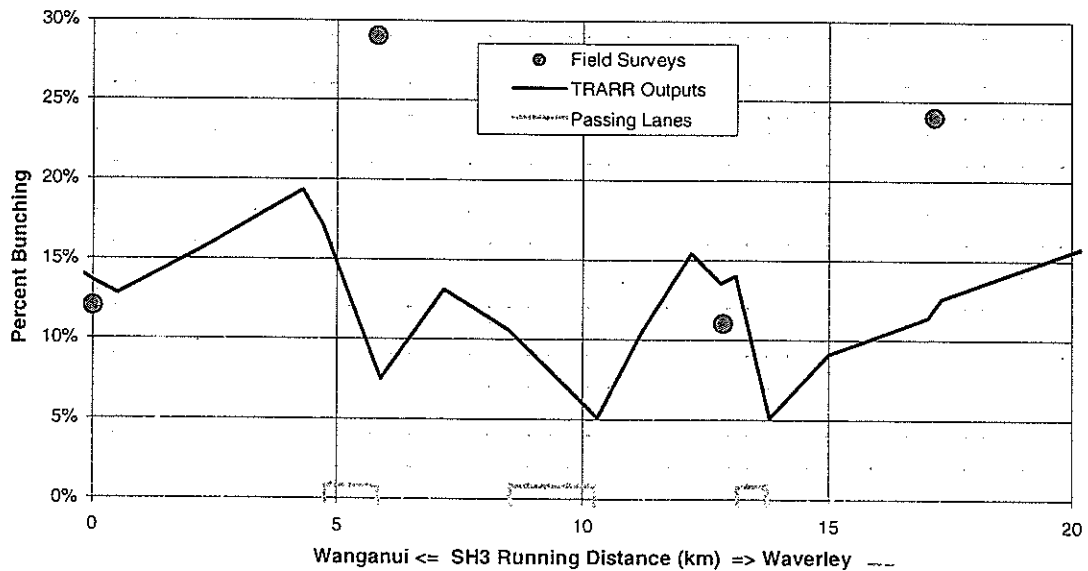
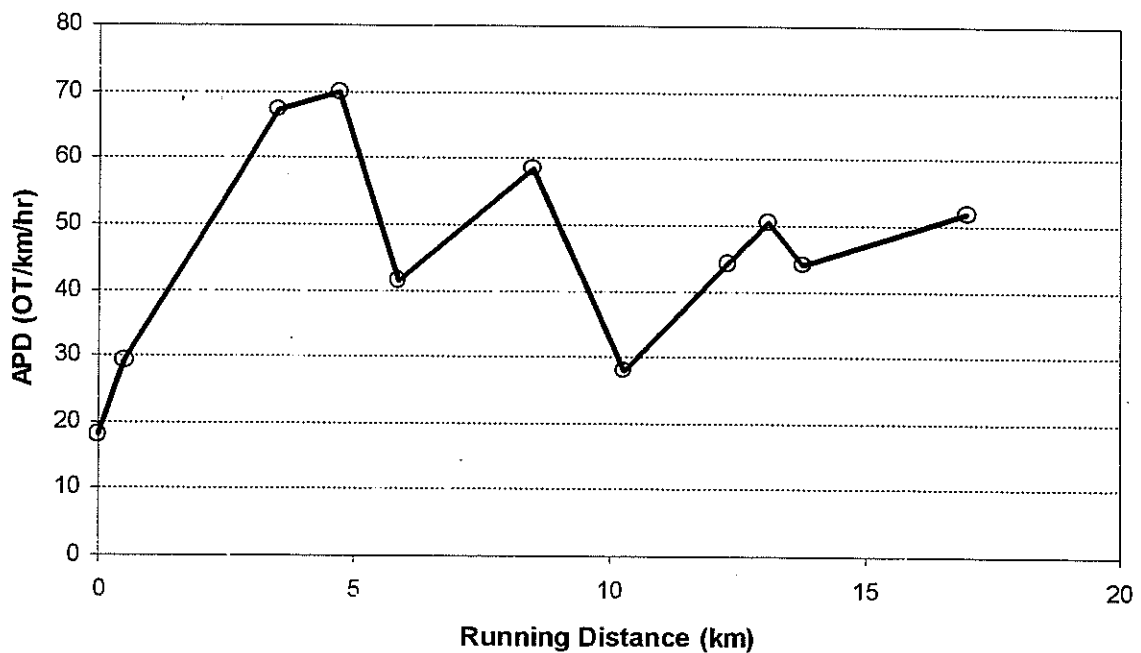


Figure 3.19 Accrued passing demand (APD) for SH3 north of Wanganui.



To check the effects on likely calculated savings, the three passing lanes were removed from both the TRARR and simplified models, and travel-time differences were compared with the previous configuration. Table 3.5 summarises the results.

Table 3.5 Comparison of travel time savings between TRARR and PEM models.

Hourly Traffic Volume (veh/hr)	Travel Time Savings per Vehicle (secs)	
	TRARR	PEM Model
50	3.9	2.5
150	20	21

The results are very encouraging and suggest that, despite some problems with the absolute representation of vehicle passing demand, the relative changes to the models are producing satisfactory travel-time benefits. Further examples would need to be tested to ensure its applicability for different road and traffic configurations.

In conclusion, the improvements have stabilised the previous difficulties over long distances whereby APD could just continue to increase even with little observed passing demand, and drop dramatically when there was a passing lane. However, given the difficulty in providing a completely satisfactory match with field and TRARR data, the use of simplified procedures for longer multiple passing-lane analyses is still questionable.

3.7 Discussion

The above investigations suggest a number of potential improvements to the existing simplified procedures. It is clear however that even the improved procedures are still not fully reflecting what is happening in the field. This may be largely a consequence of the simplified process. While additions to the model like those suggested above improve the relative fit with reality, they also take away some of the simplicity, which is an important aim to still try to retain. Fortunately software tools are available to help automate the calculations, and so such additional complexities can be incorporated relatively easily. But they do serve to reduce the transparency of the process, an important attribute when checking results for correctness.

Two key themes come through in the results identified:

- The simplified procedures still tend to over-estimate the outcomes of a given road environment. In particular, passing demand still seems to greatly outstrip supply and produce consequently high levels of APD, even though field bunching never reflects this. Conversely passing lanes seem to exaggerate the amount of passing that occurs with an equally dramatic reduction in APD not reflected in field bunching. What appears to be missing is some “equilibrium” value that a given road and traffic situation will gravitate towards. This is demonstrated by TRARR simulations where only the initial bunching value is changed. Over a long enough distance, different bunching values converge to a similar point.

- The relationship between APD and bunching is still not entirely certain, although this may be a consequence of the previous point about how it increases and decreases. The fact that passing demand within queues can increase the apparent APD somewhat complicates the issue further. The observation that practitioners can relate better to vehicle bunching than the implied passing demand suggests that maybe the simplified procedure should be re-presented in terms of %Bunching values instead (effectively by dividing by the volume).

Travel time benefits can still be determined by applying savings to the changes in vehicles bunching while, as demonstrated in Section 3.2.2, frustration benefits can also be evaluated. This is the approach that the US HCM passing lane procedures have taken and it is an attractive one.

4. Field Performance of Slow Vehicle Bays

Slow vehicle bays (SVBs) are the formalised use of very short lengths of widened, unobstructed sealed shoulder on two lane rural roads, to allow slow moving vehicles to pull out of a traffic lane and give following vehicles an opportunity to pass. Drivers of vehicles in SVBs do however have to ensure that their way is clear before they can re-enter the traffic lane. In New Zealand, SVBs are commonly used where the terrain and traffic volumes cannot justify construction of a full passing lane; and more than 70 have been built around the country on state highways, with further sites on local roads.

Previous research into driver frustration and simplified passing models (Tate 1995, Koorey et al. 1999) has been primarily concerned with the provision of passing lanes, generally of at least 800m in length (although many in New Zealand fail to meet this requirement, particularly on uphill grades). Other means of providing passing opportunities, including SVBs and wide shoulders were not specifically addressed. At the time it was recommended that, in the interim, SVB evaluations should be done using the same techniques as for passing lanes (i.e. treated like a short passing lane). However separate research to assess the performance and appropriateness of these alternative passing measures was also recommended.

To investigate these issues further, a number of approaches were taken:

- Recent overseas and local studies were reviewed to assess likely benefits from SVBs.
- Field data at a number of SVB sites were collected and analysed. This was compared with simplified models of the equivalent sections.
- TRARR models of the surveyed sites were used to confirm field data findings.

4.1 Literature Review

SVBs are short, usually less than 300m, when compared to passing lanes that should be at least 800m in length ideally. SVBs are not passing lanes but on roads with lower traffic volumes they can provide some of the benefits of passing lanes. Transit New Zealand (2000) recommends some minimum lengths for SVBs, in relation to mean traffic speed on the road in the vicinity of the bay, and these are given in Table 4.1.

Transit also recommends that SVBs should not be longer than 300m, because drivers may then treat them as a conventional passing lane. Similarly FHWA (1987) recommends that SVBs (called “turnouts” in the US) should be no longer than 600ft (190m). The implications of this in terms of performance are discussed in Section 4.4. Another recommendation by Transit is that SVBs should not be mixed with passing lanes along a route, again to minimise driver confusion. Having said that, there are instances in New Zealand where existing SVBs have been converted to exceptionally short passing lanes to meet this policy.

Table 4.1 Current minimum lengths for slow vehicle bays.

Mean Traffic Speed (km/hr)	Minimum Length (m) of SVB * (excluding entry/exit tapers)
30	60
40	60
50	70
60	80
70	100
80	135
90	175

*Minimum bay length is based on the assumption that a vehicle will enter a SVB travelling at least 8km/hr slower than the mean speed of traffic on the section of road and it will be able to stop, if necessary, within half the length of the bay while using a deceleration rate not exceeding 3m/s^2 .

4.1.1 Operational Effectiveness

SVBs can be effective in providing passing opportunities on two-lane highways, although they are not as effective as passing lanes. SVBs must be well designed and well located to be operationally effective. Harwood & St John (1985) concluded that a single well-designed and well-located SVB can be expected to provide 20% to 50% of the number of passes that would occur in a 1.6 km passing lane in level terrain. A series of well-designed and well-located SVBs can provide the operational benefits that would be provided by passing lanes. SVBs are especially well suited to mountainous terrain where the cost of providing an added lane or even a continuous paved shoulder may be prohibitive. Since they are relatively short, they can be located where they are easy and inexpensive to construct.

At sites that were surveyed by Harwood & St John (1985), the percentage of platoon leaders using the SVB ranged from 9.5% to 29.5%. The results are in agreement with the range of SVB usage (2.8% to 36%) observed by Rooney (1976).

However, a pass completed because of an SVB manoeuvre may not provide as much operational benefit as a pass completed in a passing lane. In a passing lane, the passing vehicles represent self-selected drivers with higher desired speeds than their immediate platoon leader. By contrast, at an SVB, the passed vehicles (SVB users) rather than the passing vehicles are self-selected and the passing drivers may or may not have higher desired speeds.

The passing vehicles at an SVB may simply continue downstream as a new platoon leader. Thus, an SVB may not provide as much reduction in bunching per passing manoeuvre as a passing lane as expected.

4.1.2 Safety Effectiveness

SVBs have been evaluated in two research studies in the US and have been found to operate safely. Rooney (1976) found no evidence that a significant number of crashes occur at SVBs. Sixteen SVBs in California were found to experience only one crash per 80,000 SVB users.

Harwood & St John (1985) evaluated 42 SVBs in three US states and found that a typical SVB experiences only one crash every 5 years. At seven SVBs where usage rates were observed in the field, the evaluation found only one crash per 400,000 SVBs users, an even lower rate than that found by Rooney. A safety comparison between the SVB sites and adjacent sections of conventional two-lane highway located 0.30 to 0.80 km away, found that the SVB sites had crash rates that were approximately 30% lower than the adjacent untreated sites.

Field observations by Harwood & St John found that 5% to 10% of SVB users caused a traffic conflict (such as braking by a following vehicle) when re-entering the highway from an SVB, but the crash experience associated with this manoeuvre was minimal. This finding suggests that following drivers anticipate the possible return of the SVB users to the through lanes and that their braking is a controlled response that does not indicate the likelihood of a collision. The relatively low speeds associated with many of these sites would also limit the number of actual collisions.

In 1999, Transit New Zealand commissioned Opus International Consultants (Opus) to undertake a safety review of the SVBs along SH29 over the Kaimai Ranges (Nicholson & Brough 2000). The study covered the section of SH29 between Rapurapu Road and the Ruahihi Power Station, and investigated ten SVBs. Field observations showed that 10% to 42% of overtaking manoeuvres involved perceivable vehicle braking. At most sites, only 0%-2% of overtaking manoeuvres involved the hard braking which may potentially cause crashes. The highest proportions of hard braking were found at two sites (with 10% and 18% respectively). Both these sites did not have adequate forward sight distance from the merge point, and the design of the merge area was poor. The likelihood of heavy-braking manoeuvres causing crashes was not identified.

Nicholson & Brough (2000) found that vehicles which caught up to slow vehicles near the end of the SVBs had difficulties slowing to fall in behind, and seemed almost obliged to continue on and overtake at or past the merge area. In many instances overtaking vehicles were observed to cross the centreline to pass a slow vehicle at the end of the bays. However, they recommended that, as forward sight distance was good in most cases with an opposing passing lane present and low traffic volume, this manoeuvre was not considered a major safety problem.

During the survey, a proportion of drivers were observed using the SVBs when not being followed and this may indicate that drivers may perceive them to be the same or similar to passing lanes. This perception may have implications for safety because the SVBs generally have lower geometric standards than passing lanes (i.e. more suited to lower speeds). This may be a driver education issue that needs to be addressed in New Zealand.

4.2 Field Surveys of Slow Vehicle Bays

To establish the typical use of SVBs locally, field surveys were carried out at a number of sites. These monitored the proportion of vehicles using the SVBs and amount of overtaking occurring, and the change in the proportion of bunched vehicles.

Suitable sites were selected with the following influencing variables in mind:

- Proportion of heavy vehicles and “recreational” vehicles;
- Level of existing overtaking;
- Amount of surrounding passing opportunities;
- Traffic volumes (to obtain sufficient data);
- Accessibility to surveyors.

A range of lengths, gradients and traffic volumes were also sought where possible. An initial list of sites was collected using both RAMM data and local Opus office information. Appendix A3 lists the original sites identified. From these, eight sites were selected, listed below in Table 4.2.

Table 4.2 Slow vehicle bay sites monitored in the field surveys.

Site Name	Location	SH RS/RP	Length	Grade	AADT	%HCVs
Towai	South of Kawakawa	SH 1N RP 113/10.1-10.0	90m	+2%	5200	17
Waikoau Hill	North of Whirinaki	SH 2 RP 608/5.2-5.7	100m	-7%	1600	18
Kilmog	North of Dunedin	SH 1S RP 667/14.2-14.7	450m	-10%	3900	11
Rahu Saddle	East of Reefton	SH 7 RP 152/5.9-6.0	90m	+7%	1900	20
Palmers Mill South	North of Wairakei	SH 5 RP 111/9.7-9.9	250m	+5%	3200	12
Kaimai Deer Farm	West of Tauranga	SH 29 RP 21/19.1-18.8	200m	-8%	5900	15
Old Kaimai Road	West of Tauranga	SH 29 RP 21/15.9-15.7	200m	-5%	5900	15
Cannonball Deer Farm	West of Tauranga	SH 29 RP 21/15.0-14.6	150m	-9%	5900	15

HCV – Heavy commercial vehicles.

The last three (Kaimai) sites were surveyed together to assess the effect of a series of SVBs. Figure 4.1 shows a view of the site surveyed at SH2 Waikoau Hill.

Figure 4.1 SH2 Waikoau Hill slow vehicle bay.

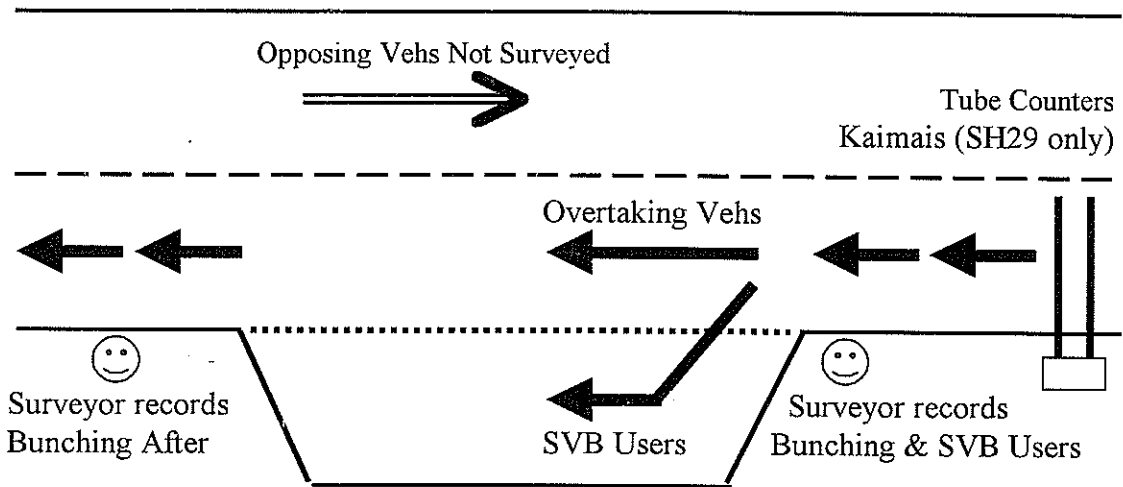


4.2.1 Methodology

Between June 2000 and January 2001, field surveys were conducted at the eight sites. Two surveyors were used at each SVB site to collect observation-based information. One surveyor recorded the proportion of following (i.e. bunched) vehicles immediately before the SVB. They also recorded the level of use of the SVB, i.e. which vehicle types were using it. Another surveyor collected bunching information at some distance after the end of the SVB. In the initial four surveys the vehicles were classified into “cars” and “trucks”; later a “recreational vehicle” category was also used for the Palmers Mill South and Kaimai sites. This last category was designed to cover the likes of campervans and towing vehicles. Previously, towing vehicles were included with “trucks”.

For the three closely sited Kaimai surveys, automated (MetroCount) vehicle classifiers were also set up ~100m before each SVB and ~500m after the last one. These measured the individual vehicle classes, speeds and headways (vehicle spacings). Figure 4.2 outlines the survey procedure diagrammatically. Survey periods lasted for a maximum of 4 hours, but for only 2 hours each at the busy Kaimai Range sites. Appendix A4 contains a copy of the survey instructions and field data sheet.

Figure 4.2 Layout of slow vehicle bay surveys.



Analysis of the data will attempt to assess the field performances of SVBs and to check if the previously prescribed simplified procedures for passing lane analysis are adequate for evaluating SVBs, with the TRARR model as a tool. TRARR has a special option to handle SVBs, referred to as the “passing bay” option. It assumes that different vehicle types have fixed criteria for deciding whether or not to use the SVBs based on the number of vehicles following them. TRARR does consider an SVB’s location but does not consider its length. The collected data from field surveys will be used to check whether TRARR is adequate for modelling SVBs.

4.2.2 Results

Survey results from the eight sites are summarised in Table 4.3, in half-hourly intervals and overall. For the Rahu Saddle site, the traffic volume was very low (about 10-15 veh/hr). The likely reason was that at Rahu the AADT is very seasonal and the tourist season was over at the time of the survey. Therefore, only an overall tally is provided for this site.

Table 4.3 Summary of survey results from slow vehicle bay sites.

Site Name	Survey Period No.	No. of Vehicles (one-way)	% Trucks, Recreation Vehicles	% Bunching Before	% Bunching After*	% Veh Using SVB	% Trucks and Recreation Veh Using SVB
Towai	1	62	9.7	41.9	44.6	8.1	33.3
	2	69	17.4	44.9	37.5	7.3	41.7
	3	66	16.7	42.7	39.7	10.6	63.6
	4	65	9.2	38.5	25.0	4.6	16.7
	5	72	19.4	34.7	38.6	9.7	42.9
	6	70	14.3	30.3	44.1	5.7	30.0
	7	69	10.1	44.9	44.1	1.5	0.0
	8	69	14.5	40.6	53.6	8.7	30.0
	TOTAL	542	14.0	40.2	40.9	7.0	35.5

Site Name	Survey Period No.	No. of Vehicles (one-way)	%Trucks, Recreation Vehicles	% Bunching Before	% Bunching After*	%Veh Using SVB	%Trucks and Recreation Veh Using SVB
Waikoau Hill	1	28	14.3	17.9	10.7	3.6	25.0
	2	25	16.0	44.4	32.0	20.0	75.0
	3	34	23.5	44.1	27.8	14.7	57.1
	4	37	29.7	18.9	7.9	8.1	27.3
	5	16	43.8	6.3	5.9	12.5	28.6
	6	45	22.2	35.6	30.4	11.1	20.0
	7	38	18.4	39.5	26.3	7.9	28.6
	8	32	31.3	18.8	9.7	6.3	20.0
	TOTAL	255	23.5	29.8	20.1	10.2	31.7
Kilmog	1	57	8.8	24.6	21.1	21.1	100.0
	2	38	10.5	44.7	21.1	15.8	75.0
	3	64	15.6	32.8	27.4	21.9	100.0
	4	66	3.0	27.3	19.2	16.7	50.0
	5	66	13.6	36.4	25.3	18.2	77.8
	6	62	8.1	37.1	25.0	17.7	80.0
	7	60	11.7	33.3	25.8	25.0	71.4
	8	62	14.5	25.8	31.7	17.7	88.9
	TOTAL	475	10.7	32.2	24.8	19.4	84.3
Rahu Saddle	TOTAL	60	25.0	1.7	5.2	40.0	80.0
Palmers Mill South	1	51	19.6	19.6	16.3	13.7	50.0
	2	69	23.2	31.9	40.6	27.5	62.5
	3	40	5.0	10.0	14.3	17.5	100.0
	4	51	17.6	21.6	17.7	21.6	88.9
	5	74	21.6	29.7	29.7	27.0	87.5
	6	78	14.1	30.8	39.0	25.6	81.8
	7	83	16.9	13.3	31.3	15.7	64.3
	8	70	15.7	18.6	35.3	15.7	81.8
	TOTAL	516	17.2	22.7	29.8	20.9	74.2
Cannonball Deer Farm	1	119	25.2	44.5	42.1	9.2	36.7
	2	117	23.9	37.6	39.0	11.1	46.4
	3	121	27.3	47.1	46.3	12.4	42.4
	4	147	14.3	45.6	47.7	7.5	33.3
	TOTAL	504	22.2	43.8	44.1	9.9	40.2
Kaimai Deer Farm	1	134	24.6	41.8	40.0	14.9	57.6
	2	100	20.0	30.0	21.9	16.0	75.0
	3	103	25.2	35.0	29.8	16.5	61.5
	4	96	16.7	30.2	32.0	12.5	56.3
	TOTAL	433	21.9	34.9	31.2	15.0	62.1
Old Kaimai Road	1	104	25.0	34.6	37.0	12.5	42.3
	2	86	26.7	48.8	43.3	14.0	39.1
	3	117	21.4	45.3	41.8	11.1	48.0
	4	115	27.0	53.9	49.2	16.5	54.8
	TOTAL	422	24.9	45.7	43.2	13.5	46.7

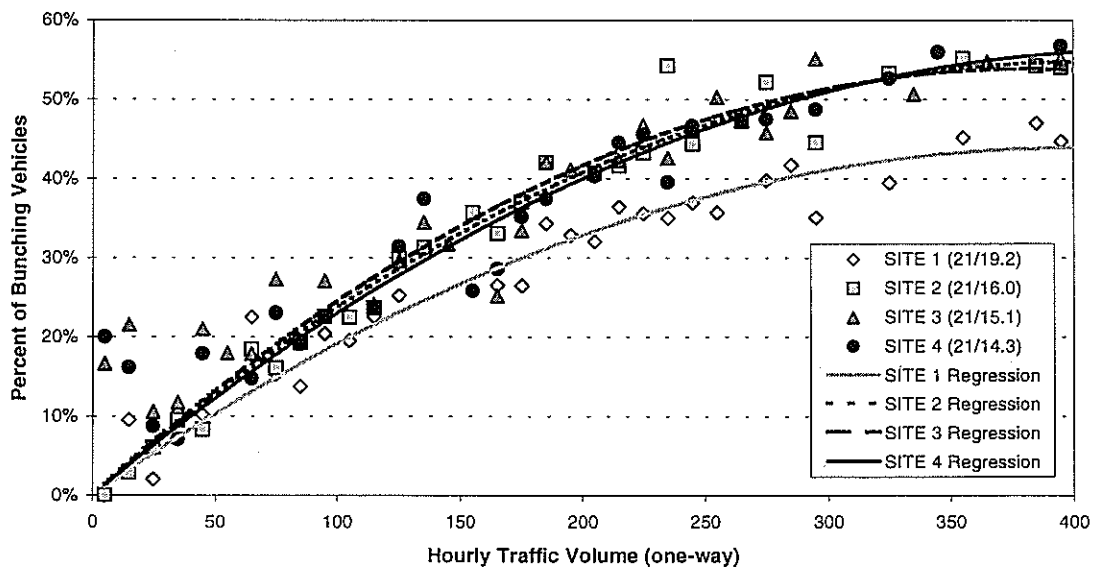
* %Bunching After values that are higher compared with %Bunching Before, are italicised.

Rather disappointingly, three out of the seven major surveyed SVBs (Towai, Cannonball Deer Farm, and Palmers Mill South) did not appear to reduce the proportion of following vehicles downstream (although only Palmers Mill showed a significant rise). However this increase may be because the downstream survey locations were not far enough away to allow overtaking vehicles to clear the overtaken vehicles. The fact that generally 10%-20% of all vehicles used the SVB suggests that the true proportion of following vehicles likely to benefit is probably greater.

Harwood & St John (1985) had also found only a 2% reduction in bunching on average immediately downstream of a SVB, and possibly up to another 4% in the following 450m. However if the alignment is fairly steep or winding downstream, bunching may not reduce any further or may even increase. The very nature of many SVB locations often provides only very short-term benefits.

This short-term benefit is confirmed by the automatic classifier surveys on the Kaimai sites (SH29). The four classifier sites provided before and after data for the three SVBs investigated, over a distance of about 5km. For each site, the level of bunching was related to the traffic volume in hourly increments. Figure 4.3 shows how the bunching rate varies at those four sites with changing traffic volumes.

Figure 4.3 Bunching levels at Kaimai SVB sites.



If these three SVBs performed in a similar manner to passing lanes, the proportion of bunching vehicles should decrease from Site 1, through Sites 2 and 3, to Site 4. However, there is no discernible trend between the latter three sites. Only Site 1 (located before all of the SVBs) is significantly different, and in fact it displays lower bunching levels than the succeeding sites. The much greater physical gap (>3km) between this site and the remaining three sites may explain this difference; and the winding alignment between the sites is likely to have caused the increased bunching levels. The remaining SVBs have only succeeded in keeping the status quo in terms of bunching.

Also to remember is that the passing vehicles are not self-selected at an SVB, and may not have higher desired speeds than the vehicles they were following. They may just simply become new platoon leaders at the downstream of the SVB.

4.3 Analysis of Results

4.3.1 Vehicle Types Using SVBs

SVBs are designed to provide space for typically slow moving vehicles, such as trucks and recreation vehicles. Although quite a few car drivers are likely to use SVBs too, this begs the question of whether SVBs are more effective where proportions of slower moving vehicles are greater.

Guidelines like Transit New Zealand's *Geometric Design Manual* (TNZ 2000) suggest that SVBs are rarely used by trucks and they are more suited to recreational vehicles and/or tourist routes where drivers of slow vehicles are usually more willing to let faster vehicles pass. Table 4.4 summarises the breakdown of vehicle types among SVB users at the surveyed sites.

Table 4.4 Vehicle types of SVB users recorded at the survey sites.

Site Name	Recorded SVB Users	Proportion of SVB Users (%)			%SVB Users Not Followed
		Cars	HCV	Rec. Vehs	
Towai	38	28.9	71.1*	*	5.3
Waikoau Hill	26	26.9	73.1*	*	15.4
Kilmog	92	53.3	46.7*	*	39.1
Rahu Saddle	24	50.0	50.0*	*	83.3
Palmers Mill South	108	38.8	26.9	34.3	43.5
Kaimai Deer Farm	65	9.2	87.7	3.1	23.1
Old Kaimai Road	57	14.0	86.0	0.0	19.3
Cannonball Deer Farm	50	10.0	82.0	8.0	2.0

* Rec. Vehs (Recreational Vehicles) were not recorded separately.
HCV – Heavy commercial vehicles.

From the sites surveyed, most of the SVB users were trucks, and in most cases they were a relatively high proportion of the total truck numbers surveyed (30%-50%). Although Kilmog and Palmers Mill South had among the highest rates of car usage in the SVBs, they also had the lowest proportion of HCVs in the traffic stream (but still the highest proportions of HCVs also using the SVB). These two sites are also quite long and may perhaps be mistaken for passing lanes. Although Rahu Saddle SVB also had a high level of car usage, this is a reflection of the very high usage overall.

As found by Nicholson & Brough (2000), quite a number of vehicles were observed using some SVBs even when no vehicles were following them. Table 4.4 also summarises the proportion of vehicles observed doing this. Rahu Saddle in particular had a very high proportion of vehicles using the SVB when not followed. No significant difference in the incidence of this behaviour was observed between vehicle types. This suggests that the road alignment may be causing this and in some

cases drivers may be mistaking it for a passing lane. Certainly the long SVBs at Kilmog and Palmers Mill have high rates of SVB users not followed. In other cases, drivers may be taking advantage of the extra road width to ease the effective curvature and travel the curve at higher speeds.

SVBs are designed to encourage slow platoon leaders (i.e. vehicles at the front of queues) to move over and let others pass. Table 4.5 shows the proportions of platoon leaders using SVBs at the surveyed sites (Rahu Saddle had only one platoon). These figures do not include those SVB users who were not followed (Table 4.4 gives more details of these).

Table 4.5 Use of SVBs by platoon leaders related to length of queues.

Site	No.Platoon Leaders	%Queues with >1veh	Proportion of Leaders Using SVB			
			Overall	Queue of 1	Queue of 2	Queue of 3+
Towai	116	43.1	27.6	30.3	23.5	23.3
Waikoau Hill	46	34.8	39.1	30.0	46.6	42.9
Kilmog	95	36.8	48.4	46.7	57.2	55.6
Palmers Mill South	75	30.7	74.7	73.1	97.0	87.5
Kaimai Deer Farm	82	42.7	57.3	46.8	86.0	82.4
Old Kaimai Road	90	55.6	40.0	32.5	60.9	60.0
Cannonball Deer Farm	116	48.3	40.5	35.0	62.1	60.9
OVERALL	621	42.7	45.4	42.4	55.1	54.9

Platoon – refers to all bunched vehicles including the lead vehicle.

Queue – refers to the vehicles behind the lead vehicle.

The overall rates are significantly higher than those found by either Harwood & St John (1985) or Rooney (1976). These may be a consequence of the survey observing better designed SVBs, or a more prevalent habit in New Zealand of using SVBs. The figures in Table 4.5 also show that drivers with more than one vehicle following them are more likely to use a SVB, but no distinction between queues of two and queues of three or more is apparent.

4.3.2 Vehicle Bunching Near SVBs

The Borel-Tanner Distribution (BTD) provides a reasonable model for describing the distribution of bunch (or platoon) sizes in traffic on two-lane, two-way rural roads. The probability of a bunch size r is given by

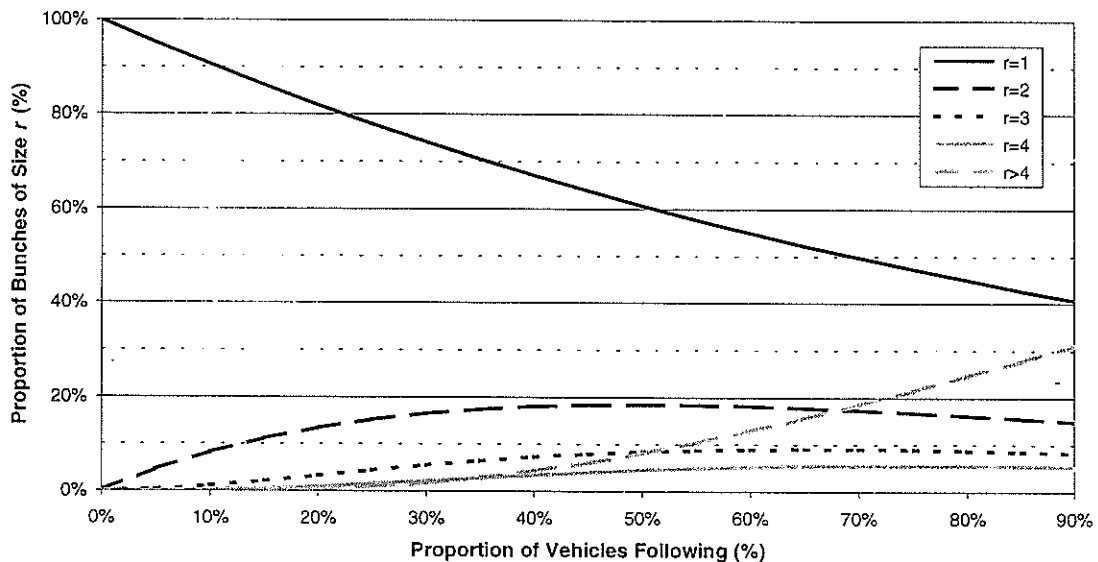
$$P(r) = [rz \exp(-z)]^{r-1} \frac{\exp(-z)}{r!} \tag{19}$$

for $r = 1, 2, 3, \dots$ and where z is the proportion of vehicles following. This is also related to the mean bunch size $m = 1/(1-z)$, or alternatively, $z = (m-1)/m$.

Analyses of the Towai, Waikoau Hill and Kilmog sites found correlations of better than $r^2 = 99\%$ between the observed distribution of bunch sizes (both before and after the SVB) and that predicted by the BTD. On this basis, further analysis has been done using the BTD to model bunching at SVBs in general.

The expected proportion of various bunch sizes for various overall vehicle-following rates can be derived using the BTD and plotted, as shown in Figure 4.4. For comparison, from the previous data presented, the typical vehicle-following proportions observed at the surveyed sites were between 20%-40%. Also note that the plot shows the relative proportions of bunches, not vehicles. At 90% following for example, although 40% of bunches are lone vehicles, they make up only about 4% of all vehicles because of the number of very large bunches present at this level.

Figure 4.4 Distribution of vehicle bunch sizes related to proportion of vehicles following.

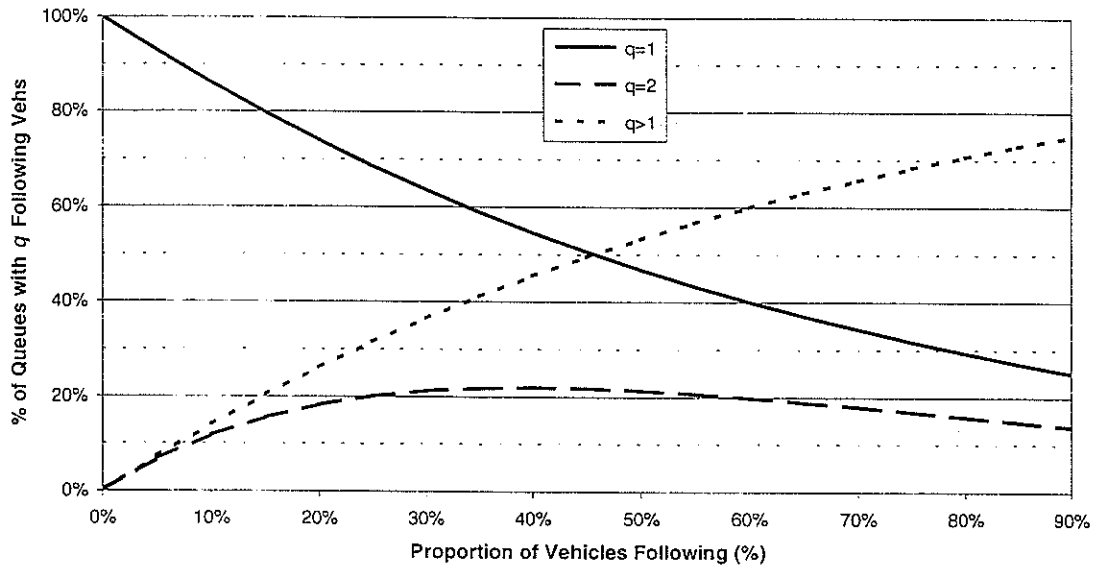


Single (lone) vehicles are shown to comprise the greatest proportion of “bunches”; and in fact they make up the majority of bunches when less than 70% following overall is observed. Hence at a given point, a great number of vehicles are not likely to need a SVB to pass another vehicle. In terms of actual numbers of vehicles involved however, single-vehicle bunches only involve the majority of vehicles up to about 32% following. Also significant is the gradual climb in “long” bunches (i.e. greater than 4 vehicles), and in terms of overall vehicles they actually comprise the majority from about 55% following upwards.

If single vehicles are ignored and we examine the make-up of bunches with following queues, a different picture emerges. Figure 4.5 shows the proportion of queues with one or two vehicles following respectively, and the proportion for all queues of more than one vehicle.

The results highlight that queues of only one following vehicles are the majority only when less than 45% following overall is observed. In terms of vehicle numbers, drivers are in fact more likely to find themselves in a multi-vehicle queue from about 23% following. Therefore queues of two or more vehicles-following play a significant part when considering the operation of SVBs.

Figure 4.5 Distribution of vehicle queue sizes to proportion of vehicles following.



4.3.3 Minimum Length for Slow Vehicle Bays

In current New Zealand guidelines (Transit NZ 2000), the minimum length for SVBs is based on the assumption that the vehicles entering the bay can stop safely if necessary. It is evident however that many drivers do not want to slow down significantly in SVBs, given the number of potential conflicts observed at the merge taper. This may partly be related to some drivers who feel that, as they have conceded to other traffic sufficiently, it is now their “right” to be able to re-enter the traffic stream without delay. Still others may be under the impression that the merge at the end of the SVB is similar to a passing lane in that no lane has priority over the other.

The use of SVBs may also be affected if drivers do not feel that they can maintain their momentum in the length available while allowing following vehicles to pass safely. This has implications for assessing travel time benefits of SVBs. Drivers may not want to enter an SVB that slows them down inordinately. Conversely, those who do use the SVB may lose significant time that will cancel out the time savings achieved by the overtaking vehicles.

If we change the length assumption so that vehicles entering the bay can still travel at their own speed with little delay, then we can calculate the new minimum length required. Table 4.5 above showed that about 50%-70% of platoon leaders were followed by one vehicle, with the remaining 30%-50% followed by at least two vehicles. Therefore it is pertinent to examine the lengths required to pass at least one or two vehicles.

Consider a slow vehicle, travelling at u km/h, being followed by a vehicle (or vehicles) that would like to travel at v km/h. The relative distance required to overtake the lead vehicle is d m (from a point behind the lead vehicle where the overtaking begins to a point in front where the overtaking is completed). While this gain is being made on the overtaken vehicle, the overtaking vehicle will be travelling forward at its desired speed. (Although this slightly under-estimates the acceleration required to reach this speed, it is probably compensated somewhat by a slightly higher speed while overtaking.) Therefore the required road distance, L , to complete this manoeuvre is:

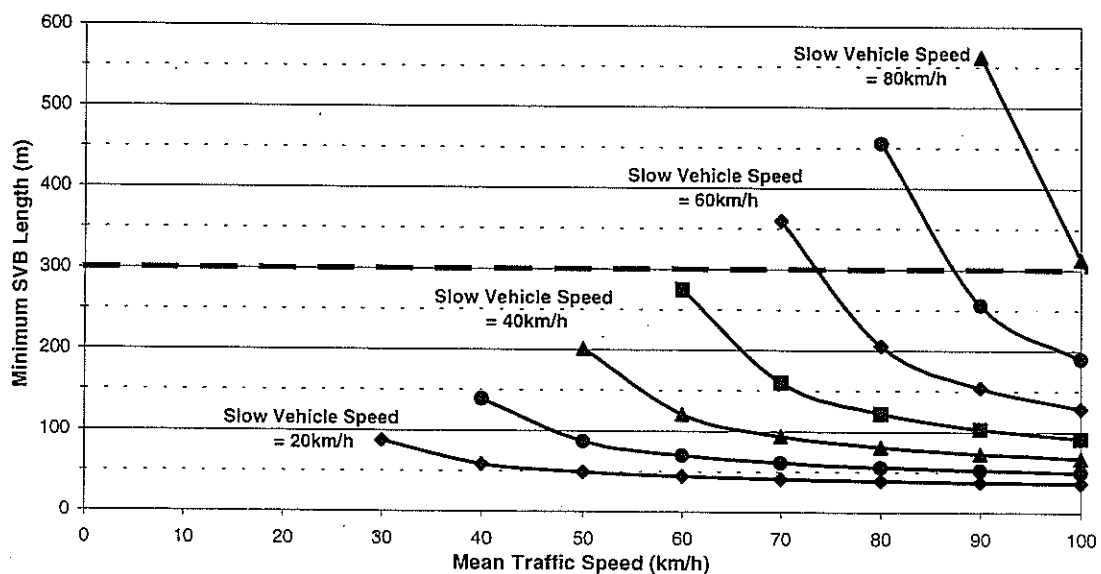
$$L = d.v / (v-u) \tag{21}$$

A reasonable distance d might be to allow a one-second clear gap either side of the overtaken vehicle (although two seconds is recommended in normal flow, because overtaking vehicles tend to close in beforehand), plus the length of the vehicles concerned. From the survey data above it is reasonable to assume, say, a 6m car overtaking a 12m truck, will require a distance of:

$$d = u/3.6 \times 2 + (6 + 12) \tag{22}$$

So, for example, a vehicle wanting to travel at 70km/hr to overtake a vehicle travelling at 50km/hr would require $(50/3.6 \times 2 + 18) \times 70 / (70 - 50) = 160.2$ m to overtake without impeding the progress of the slower vehicle. Figure 4.6 plots the required distances for various combinations of desired mean traffic speeds and slow vehicle speeds.

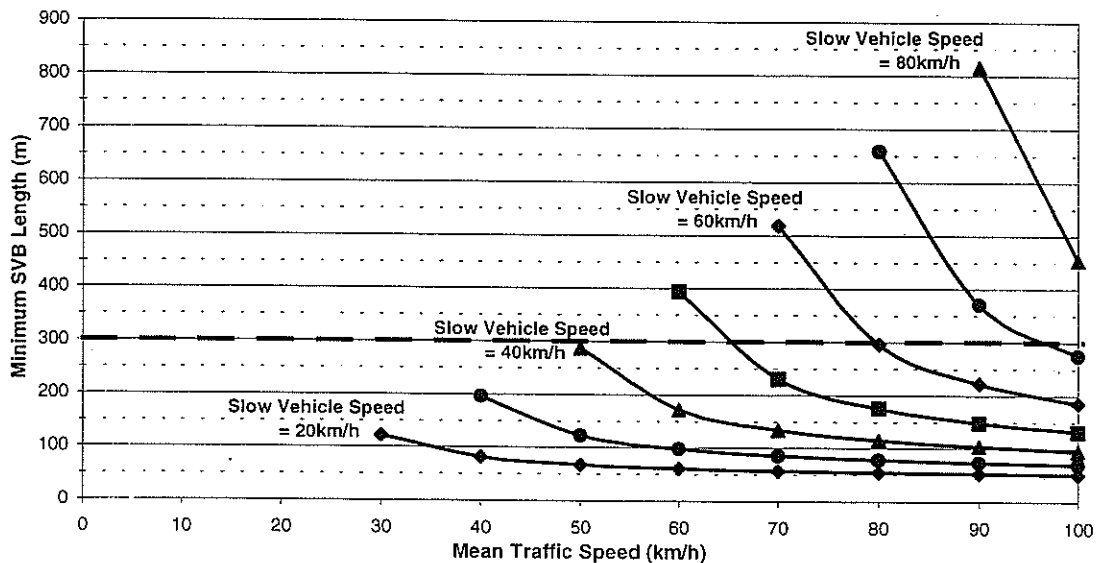
Figure 4.6 Minimum length of SVB, for one vehicle to overtake.



Note that for high-speed situations (mean traffic speed >60km/hr), with little difference in vehicle speeds (i.e. <20 km/hr), the minimum required length is greater than the recommended maximum of 300m.

If two following vehicles wanted to overtake the lead vehicle safely, then a longer distance d would be required, to allow for the extra vehicle length and clear gap. Figure 4.7 shows the results from such calculations.

Figure 4.7 Minimum length of SVB, for two vehicles to overtake.



Now an even greater minimum length is required, with many situations (having traffic speeds greater than 50km/hr) requiring lengths greater than 300m. Therefore if an SVB is limited to a 300m length (as recommended), there will be instances where the overtaken vehicle has to slow down or stop to avoid a conflict at the merge taper. For example, the mean traffic speed in the area of the three Kaimai SVBs is about 87km/hr, and their lengths are between 200m and 300m. Given the traffic volumes present there, a driver using any of the SVBs will probably have to give way to the following traffic at the merge taper.

Table 4.6 summarises the above findings in a form similar to the existing recommended lengths (refer to Table 4.1). The 10km/hr speed differential used is slightly greater than the 8km/hr used previously, which however serves to minimise the relative increase in lengths over the previous table.

Application of these values would require estimation of the likelihood of queues of more than one vehicle being present in the traffic stream. This can be determined from field surveys on-site and compared with the theoretical proportions derived in Section 4.3.2. In some cases on steeper gradients it may be possible to use a greater speed differential to reflect the two conflicting traffic streams, in which case the minimum lengths from Figures 4.6 and 4.7 could be used.

Table 4.6 Recommended minimum SVB lengths.

Mean Traffic Speed (km/hr)	Minimum Length of Slow Vehicle Bay (m)*	
	followed by 1 veh	followed by 2 veh
30	90	120
40	140	195
50	200	285
60	275	395
70	360	520
80	455	660
90	560	815

*Minimum bay length is based on the assumption that a vehicle will enter a SVB travelling at about 10km/hr slower than the mean speed of traffic on the section of road and it will **not** be delayed.

To maintain the adequacy of the 300m maximum recommended length, the above findings would suggest that SVBs should only be located where the mean traffic speed is less than about 60km/hr. Where traffic volumes are greater and longer queues are more likely, even lower mean traffic speeds are desired before considering a SVB.

4.4 TRARR Models

An option for handling SVBs is incorporated into TRARR (known as “passing bays” in TRARR terminology). Though TRARR does not consider the length of an SVB and treats it as a stop, it may still provide reasonable output because most SVBs are generally very short. To test the validity of TRARR modelling of SVBs, two models were built to simulate the Waikoau Hill and Kilmog SVBs (they are the only two SVBs that effectively reduced the platooned vehicles). The two sites provide an interesting contrast in SVB lengths, being 100m and 450m respectively. The site models were calibrated and validated with the data from field surveys.

SH Road Geometry data was used to create a TRARR road file surrounding each site. TRARR was then run using the same volumes, %HCVs and initial %Following as observed in the field. The downstream %Following was then compared with the observed field data.

TRARR uses a PBAYS file to specify the location of and parameters associated with SVBs. Initially the default parameters provided by ARRB were used for modelling but they gave a poor fit with the observed data. Inspection of the PBAYS file revealed that very low numbers of vehicles were specified to use the SVB. In particular some heavy vehicles would never use the bays, while other vehicles would only use them when they had a number of vehicles queued behind them. Based on the field survey findings, some adjustments were made to the PBAYS parameters, resulting in a far better fit. A copy of the updated PBAYS file is contained in Appendix A.5 and it is recommended that this updated version be used for SVB modelling in New Zealand. Interested practitioners are welcome to contact the authors of this report for an electronic copy.

4. *Field Performance of Slow Vehicle Bays*

The outputs from TRARR models are compared with the survey results in Tables 4.7 and 4.8. The differences are acceptable, and hence we can say that the SVBs modelled by TRARR provide a realistic reduction in the proportion of platooned vehicles.

Table 4.7 Comparison of Waikoau Hill SVB data with TRARR Model.

Obs No.	One Way Flow (vph)	%HCV	%Following at Start	%Following at End		%Diff
				Field	TRARR	
1	56	14.3	17.9	10.7	14.6	+3.9
2	50	16.0	44.4	32.0	30.4	-1.6
3	68	23.5	44.1	27.8	34.2	+6.4
4	74	29.7	18.9	7.9	13.7	+5.8
5	32	43.8	6.3	5.9	3.3	-2.6
6	90	22.2	35.6	30.4	27.3	-3.1
7	76	18.4	39.5	26.3	31.2	+4.9
8	64	31.3	18.8	9.7	13.2	+3.5
OVERALL			29.8	20.1	22.5	+2.4

Table 4.8 Comparison of Kilmog SVB data with TRARR Model.

Obs No.	One Way Flow (vph)	% HCV	%Following at Start	%Following at End		%Diff
				Field	TRARR	
1	114	8.8	24.6	21.1	21.4	+0.3
2	76	10.5	44.7	21.1	33.8	+12.7
3	128	15.6	32.8	27.4	27.2	-0.2
4	132	3.0	27.3	19.2	25.3	+6.1
5	132	13.6	36.4	25.0	29.4	+4.4
6	124	8.1	37.1	25.8	30.6	+4.8
7	120	11.7	33.3	25.4	26.1	+0.7
8	124	14.5	25.8	31.7	22.7	-9.0
OVERALL			32.2	24.8	26.1	+2.0

However, TRARR models SVB as an extra stop where the slow vehicles can pull aside and stop while being overtaken. They will join the traffic again only when there is no vehicle behind. Hence the travel-time savings for those overtaking vehicles are greatly offset by the delays experienced by those SVB users. In reality, some of those vehicles should be able to travel at their own speed in the bay without delay while being undertaken. Therefore, TRARR under-estimates the actual travel-time savings.

One solution to assess travel-time savings may be to model the SVB as a short passing lane. However this is likely to over-estimate travel-time savings. The true answer is probably somewhere between these two values and will be dependent on the likelihood of overtaken vehicles having to slow down or stop. The previous discussions on SVB use and minimum SVB lengths should be applied to assess this likelihood and to derive a realistic time saving.

4.5 Simplified Procedure in PEM

In the current *Project Evaluation Manual* (PEM, Transfund 1999), the benefits of SVBs are evaluated by the same simplified procedure used for passing lanes. However, an SVB generally cannot provide as many passes as a passing lane (although it usually achieves a better rate per length). Neither can it provide as much reduction in the proportion of platooned vehicles as a passing lane. On the other hand, the SVB users will be delayed when they have to give way to the following vehicles at the exit of the SVB. Therefore, using the simplified procedure for passing lanes will probably over-estimate the benefits provided by SVBs.

When the traffic volume is very low, the SVB users have little chance to be delayed. However, with such low volume and small amount of users, the bay does not provide many travel-time savings at all. When the traffic volume is high, the SVB users are very likely to be caught up by the following vehicles and delayed at the exit of the bay. Those delays offset the travel-time savings of the overtaking vehicles. Hence we cannot expect much overall travel time benefits by the SVB.

Therefore, claiming only the frustration benefits, as measured by the reduction in the percentage of bunching vehicles (PEM), and possibly some safety benefits, may be more realistic and reasonable.

4.6 Simplified Modelling of Frustration Benefits

Assessment of frustration benefits requires determination of the change in vehicles following (or bunched). As seen above, this change can be reasonably assessed by TRARR. However the effort required to do this may not be justified for many small SVB projects. The simple nature of SVB interactions allows a theoretical approach to be developed instead.

At an SVB, where the bunching proportion at the entry is a (as measured in the field), the remaining proportion must either be leading a bunch or be on their own. The number of bunches, including isolated single vehicles, is therefore equal to

$$(1-a) \times Volume \quad (22)$$

Assuming that vehicle bunching can be modelled using BTD as discussed in Section 4.3.2, the probability of a “bunch” of size 1 (i.e. single vehicle) is given by:

$$P(1) = e^{-a} \quad (23)$$

Therefore we can estimate that the number of bunches, excluding those of size 1, is

$$(1-a) \times Volume \times (1 - e^{-a}) \quad (24)$$

Assume the proportion of those platoon leaders who would use an SVB is s , with the result that the vehicle immediately following them would no longer be bunched. Therefore the number of vehicles freed up are:

$$(1-a) \times Volume \times (1 - e^{-a}) \times s \quad (25)$$

The effect on the overall bunching proportion can be seen by dividing by the volume. Therefore the bunching proportion, b , at the end of the bay can be estimated by:

$$b = a - [(1-a) \times (1 - e^{-a}) \times s] \quad (26)$$

This formula relies on the overtaken vehicle not continuing to be part of the following queue, on the assumption that all the following vehicles have faster desired speeds. This may not necessarily be the case if some following vehicles had similar desired speeds and were quite content to follow. However it should provide a reasonable approximation of bunching reduction.

From the field survey, we know that on the average 45.4% of platoon leaders would move to the SVB and let the following vehicles pass (see Table 4.5). By applying this figure to the formula above, the bunching rates at the end of the Waikoau Hill and Kilmog SVBs can be calculated. They are compared with the survey results in the following Tables 4.9 and 4.10. Note that the on-site usage rates at these two sites differed slightly from the average, but affect the overall values very little.

Table 4.9 Comparison of Waikoau Hill SVB data with simplified bunching formula.

Survey Period No.	%Following at Start	%Following at End		%Diff
		Field	Formula	
1	17.9	10.7	11.8	+1.1
2	44.4	32.0	35.3	+3.3
3	44.1	27.8	35.0	+7.2
4	18.9	7.9	12.6	+4.7
5	6.3	5.9	3.7	-2.2
6	35.6	30.4	26.8	-3.6
7	39.5	26.3	30.5	+4.2
8	18.8	9.7	12.5	+2.8
OVERALL	29.8	20.1	21.6	+2.8

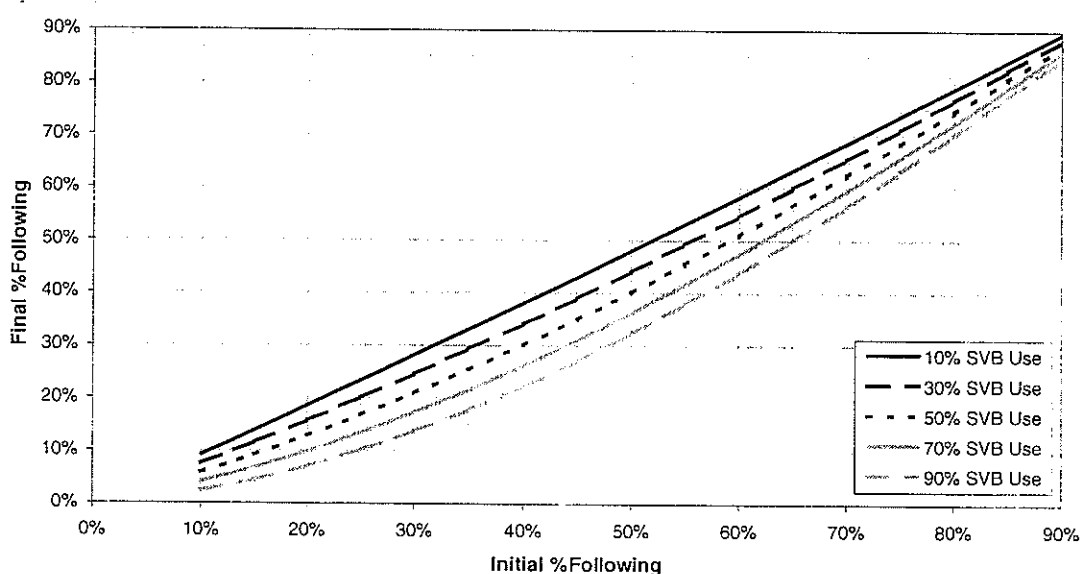
Table 4.10 Comparison of Kilmog SVB data with simplified bunching formula.

Survey Period No.	%Following at Start	%Following at End		%Diff
		Field	Formula	
1	24.6	21.1	17.1	-4.0
2	44.7	21.1	35.7	+14.6
3	32.8	27.4	24.3	-3.1
4	27.3	19.2	19.4	+0.2
5	36.4	25.0	27.6	+2.6
6	37.1	25.8	28.2	+2.4
7	33.3	25.4	24.7	-0.7
8	25.8	31.7	18.1	-13.6
OVERALL	32.2	24.8	23.7	-0.9

The results above show that the developed formula is adequate for predicting the bunching rates after an SVB. Therefore, it can be applied to the calculation of the frustration benefits from a proposed SVB. The parameter s (the proportion of those platoon leaders who would move into an SVB) cannot be obtained directly since the SVB is not built yet. However, a value can be estimated either from average values collected from other similar existing SVBs in the same area or based on the findings in this report.

To assist in determining the likely improvement, a range of SVB use rates have been applied to various initial bunching rates to determine the likely reduction in bunching. Figure 4.8 shows the results, allowing easy interpolation.

Figure 4.8 Theoretical improvements in %Following at SVBs.



4.7 Discussion

SVBs are evidently well used in New Zealand, especially compared with some overseas studies. Their use however appears to be rather dependent on the location and design of each site, in that a poorly placed or insufficiently long SVB can suffer low usage and provide little benefit.

The eight sites surveyed represent above-average traffic volumes compared with the average AADT of about 2500 vpd obtained from the original 70 SVBs identified. Even this lower volume is above the recommended maximum of 2000 vpd (TNZ/LTSA 1997). The findings however generally have broad enough trends to be applicable across a wider range of volumes. Also the higher volume sites can often best justify construction of SVBs.

There still appears to be some confusion among drivers between the intended use of passing lanes and SVBs. This is seen in the number of vehicles using an SVB even when no one is following them, and with many of the conflicts observed at the merge tapers. This confusion may be related to the relative length and appearance of some SVBs, which can give the impression of being a passing lane in terms of road markings for example.

Roading authorities should ensure that SVBs and passing lanes are marked quite distinctly and that there are no “hybrid” layouts. Some general driver education (e.g. via LTSA’s “Code Red” programme) would probably also be of benefit. Alternatively, more targeted education of the main users, i.e. heavy vehicle drivers, could be more beneficial, e.g. via the Road Transport Association newsletters.

One way to more clearly distinguish SVBs from passing lanes would be to set stricter limits on their respective maximum and minimum lengths. For example, by enforcing that SVBs never extend beyond 300m, and that passing lanes are never shorter than 600m, a clear dichotomy would be apparent to the road user. The practicality of this is the only concern, given the terrain that roading designers are sometimes faced with when locating passing opportunities. Modifying all existing passing opportunities that did not meet these criteria would also be needed, for example by extending, re-marking/signing, or even removing them.

It is unclear from the results whether SVBs provide more benefits downhill than uphill. The uphill sites surveyed however did not appear to be very successful in reducing bunching. The key effect will be when assessing the required minimum length for safe passing, because the greater speed differentials that are likely on uphill gradients can allow shorter SVBs to be used. Conversely vehicles may be more tolerant of a shorter downhill SVB, knowing that gravity can get them moving again more easily should they have to slow down or stop. (Of course this can count against them when they are trying to stop.) It may be that “summit” SVBs provide the best of both worlds by allowing for a greater speed differential on entry uphill, while providing for easier stop-start merges on the exit downhill.

SVBs may produce rather different benefits than that of passing lanes. This has mainly to do with the requirement for SVB users to yield to passing vehicles in the main traffic stream. For a short SVB with significant queuing, this may mean that any travel time benefits gained by the passing vehicles are negated by the delay placed on the overtaken vehicle.

As a result, project evaluation of SVBs would be best served by adopting a different process to those available for passing lane evaluation. In particular, the first step required is to determine how well the proposed SVB will work in providing safe passing opportunities without unduly impeding the overtaken vehicles. A short SVB relative to the queuing demand is likely to produce benefits only through reduction in frustration and possibly crashes. Only a sufficiently long SVB is likely to also provide travel time savings, which could be assessed using either TRARR or the existing simplified passing lane model.

The safety benefits of SVBs are not entirely clear. Certainly, the Kaimai observation surveys (Nicholson & Brought 2000) have shown that a poorly designed SVB, especially with regard to the merge area, may cause an unacceptably high level of serious traffic conflicts. The effect of the SVB length on this behaviour should also be considered. Overseas crash studies have been inconclusive, but these kinds of site deficiencies may be masking true crash savings achieved when drivers are able to pass and frustration is reduced.

Unlike many passing lanes, SVBs may not have as great an influence on traffic operations downstream. Partly because many are built in winding terrain, vehicles that can overtake at SVBs may soon find themselves in another queue. Therefore crash savings beyond the site may be very limited. At the site, crash savings are likely if, before the SVB was constructed, drivers attempted to overtake here anyway (e.g. because of the available sight distance, or the presence of a previous wide shoulder), and thus the new SVB provides a safer alternative.

5. Development of Framework for Detailed Modelling

The nature of New Zealand's terrain and traffic volumes mean that drivers here will continue to rely on two-lane rural highways with intermittent passing-lane/four-lane sections for many years to come. To assist prioritising future work for funding, Transfund wants an accurate assessment of the benefits attributable to works such as realignments and passing lanes. To date this has generally required detailed modelling for these projects, especially for those beyond their initial scoping stage. This is usually by means of a rural road simulation model, such as ARRB Transport Research's TRARR package (Shepherd 1994). This is a relatively specialised tool and, despite recent improvements in its data collection requirements, it is still fairly time consuming (and hence costly) to make use of.

In recent times, a "simplified procedure" has been developed for Transfund that enables an assessment of the benefits for "simple" passing lane projects (Transfund 1999). This was based on the findings from Stages 1 and 2 of this research (Tate 1995, Koorey et al. 1999). Other parts of this Stage 3 research have looked into improving these procedures. Transit NZ has also been independently developing an alternative simplified procedure that will allow a more strategic approach to be taken when planning and prioritising passing lane improvements along routes, without detailed analysis of each site (BCHF 2000).

Although these simplified procedures are helping to ease the effort required to evaluate and authorise passing opportunity improvements, there are limitations to this approach. In particular, evaluations of projects involving realignments are still difficult to assess, and the effects of multiple passing lanes (or continuous four-laning) over long lengths of road are also more susceptible to wide variations in results. For these reasons at least, simulation would appear to provide more accurate information. Any move to "simplified" methods, as developed for passing lanes, will still require research using simulation to confirm the validity of the methods for general use.

The predominant tool in New Zealand for rural simulation is TRARR, but there are growing concerns about its applicability to all local scenarios requiring evaluation. The practical limitations of TRARR are also a concern, especially given the lack of support by its publishers. What has perpetuated its use here is the lack of feasible alternatives in New Zealand.

A US software package, TWOPAS (St John & Harwood 1986), which is similar to TRARR in operation, holds some promise as a possible replacement. Another approach is to consider the feasibility of using more recent urban/motorway micro-simulation tools like PARAMICS (SIAS 1999) for rural modelling work.

This section reviews the features and limitations of existing detailed rural simulation models for use in New Zealand, and develops a framework for an improved tool that would better meet our needs. The existing features and deficiencies of TRARR and

other common simulation tools world-wide are summarised and compared. This is based on literature review, correspondence with relevant organisations (e.g. Transit NZ, ARRB), and the personal experience of the researchers.

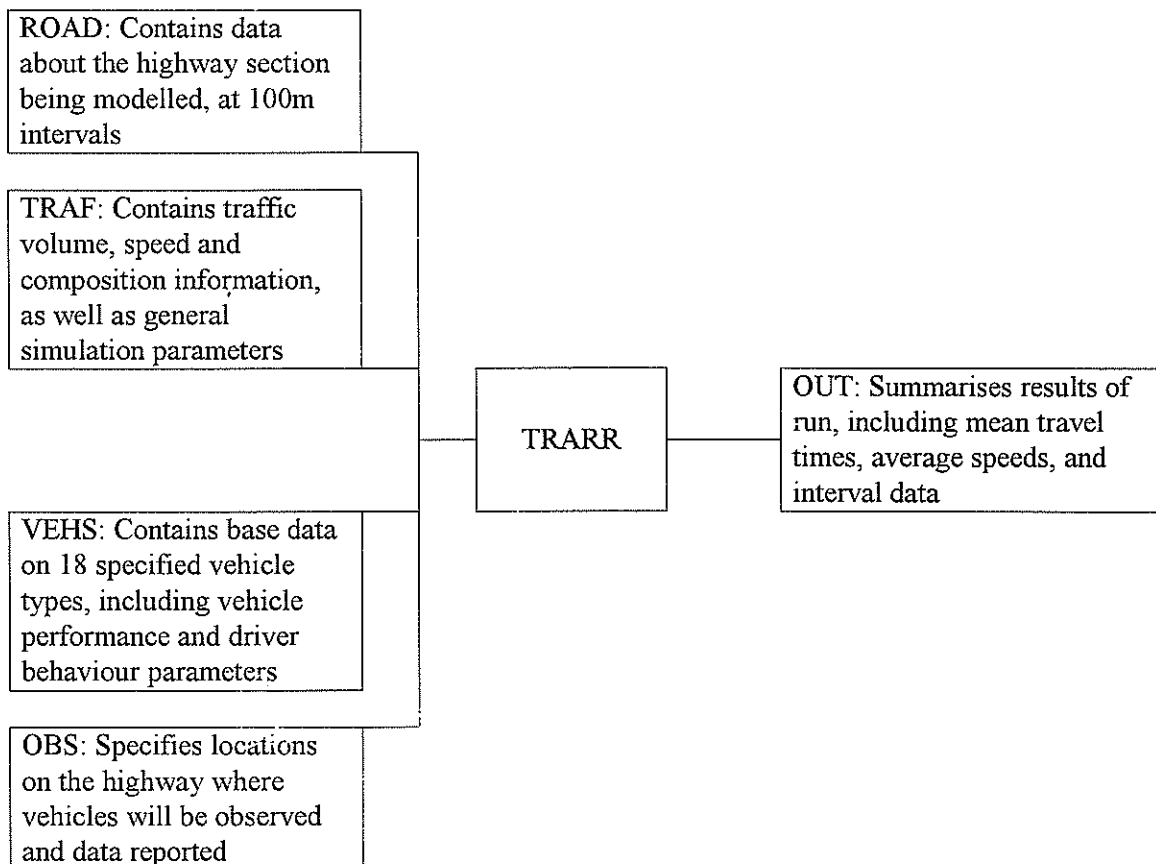
From this, a framework of ideal features for New Zealand rural detailed modelling is established. It is envisaged that the resulting prescription could form the basis of a specification brief, should future development of such a modelling tool be undertaken.

5.1 Existing Features and Limitations of TRARR

TRARR was originally developed in the late 1970s and 1980s by the ARRB. Originally run on mainframe computer systems, the program was ported to a PC version (3.0) in 1986. The most recent version (4.0) was produced in 1994 and included a user-friendly interface (albeit with reduced functionality) and the ability to import road geometry data for the creation of road sections. The latter greatly simplified the data creation requirements, particularly as New Zealand state highways had been surveyed using the Road Geometry Data Acquisition System (RGDAS) in 1992.

The basic structure of TRARR’s input and output has remained largely unchanged throughout the various versions. Figure 5.1 outlines the main data files involved.

Figure 5.1 Data files used in TRARR.



TRARR is largely based on deterministic modelling, i.e. it uses fixed parameters for various inputs (e.g. vehicle power, overtaking behaviours, etc.). More use of probabilistic or stochastic modelling (whereby parameters can take on a number of random values within a distribution) may improve the reality of the outputs, although it is not clear whether this additional complexity would produce significant benefits.

As a modelling tool for this purpose, TRARR has proved to be an adequate package. However a number of potential drawbacks have been identified through practical experience, limiting TRARR's use for all rural road simulation work in New Zealand. Some of these issues are discussed below. The desired improvements are not likely to be seen in the TRARR program in the future, as ARRB have indicated that they are not planning further development.

5.1.1 Handling Side Road Flows

An assumption of TRARR when running a simulation is that all vehicles must enter the model at one end and depart at the other, i.e. the traffic volume is constant throughout. No consideration is given for any significant side road flows that may impact upon the main route along the way. This restriction means that road sections modelled in TRARR currently have to be limited to those between major junctions or other locations (e.g. urban areas) that may affect significantly traffic volumes. In some cases this may place an unreasonable restriction on what can be adequately modelled. Although a road can be split up into multiple models to reflect changes in flows, this is an additional effort to set up, calibrate and analyse.

Another factor to consider is that the presence of side roads often affects the decision of drivers to overtake in certain places. Although it is possible to place no-overtaking restrictions at the intersection sites, it is not clear how well this method models the true behaviour of overtaking near intersections.

5.1.2 Restricted Speed Zones

Another assumption of TRARR is that vehicle speed is only constrained by road alignment and interaction with other vehicles. This implies that any short speed-restricted areas that vehicles may travel through (e.g. small towns) must be excluded or ignored. This is often impractical for many New Zealand highway sections, along which are many small towns. One alternative approach used by the authors (Opus 1999) is to manually adjust the speed parameters ("road speed indices") of the road to reflect the relative change in speed limits. This is however an artificial contrivance that can only partly take advantage of how TRARR currently operates.

Similarly no consideration is taken within TRARR of the effects of road widths and roadside development on speed selection. Again the TRARR documentation (Hoban et al. 1991) has the suggestion that users could modify the road speed indices to achieve this. However this relies on the users knowing what are appropriate values to use, a task probably better achieved if the values were standardised into the model.

5.1.3 Highly Congested Situations

TRARR is largely designed for the modelling of low to moderate flows on two-lane rural roads. This is partly because of the assumption that roads with relatively high traffic flows would probably be four-laned and to a high alignment standard, and hence not require modelling of improvement works. However in New Zealand this is not always the case (e.g. consider the largely two-lane highway between Auckland and Hamilton where flows of >1000 veh/hr are prevalent). As a result, although TRARR can model high volumes, the results are likely to be less realistic when the traffic flows become less stable.

A possible example of this effect of high traffic volumes can be seen in passing lane closures during peak holiday periods. This interesting concept has been applied at a number of passing lanes on major highways to improve overall traffic efficiency (Transit NZ 2001). When the traffic is heavier than usual, hold ups generally occur as traffic breaks off at a passing lane and proceeds to merge, slowing down all traffic. Removing the passing lane (by closing it off with traffic control and cones) means traffic will not have to merge and can move along in a steady stream.

One of the sites used by Transit was the SH1 Otaihanga passing lane, surveyed as part of task 2 of this research. Therefore, for comparison, a TRARR model was set up to model both scenarios (with and without passing lanes) at traffic flows up to 1800 veh/hr in the direction of the passing lane. The results however mirrored typical findings from lower volumes, whereby the option with the passing lane included provided better travel times. In fact at 1800 veh/hr, removing the passing lane appeared to dramatically worsen the situation. TRARR did not appear able to adequately identify the benefits seen in practice when the passing lane was closed. This may be a consequence of the default values specified in TRARR for end-of-passing-lane merging, and without further investigation the results are inconclusive.

Congested situations also occur when road works are imposed on a road, particularly where lane closures are involved. Although less critical, it would be desirable to be able to model these as well for planning for temporary traffic management purposes.

5.1.4 Difficulty in using Field Data for Calibration

As with most traffic models, TRARR requires calibration of modelled outputs to accurately match actual observed outputs. This can be a difficult process to do properly, particularly for long road sections. As well as producing comparable overall travel times, TRARR should also produce similar spot speeds and bunching levels to those measured at various points along the highway.

TRARR requires the user to assess what changes are required to the input file parameters to achieve better calibration, with little guidance available. An improved package would be able to take field data as inputs to perform some self-calibration where possible.

5.1.5 Software Operational Issues

A number of other practical problems exist that, while not affecting the integrity of the TRARR model, do impact on the efficiency of running it.

TRARR is a DOS-based program and thus lacks a modern interface similar to those we have become accustomed to in more recent software. A “Windows-like” graphical front-end (still DOS-based) was produced in TRARR 4.0, but many people prefer to use the underlying TRARR “engine” on its own because of greater flexibility with modelling options. This still presents some operational difficulties, particularly for those less familiar with DOS environments, and can present compatibility issues when run under Windows.

One of the big advantages of TRARR 4.0 was the ability to import road data from RGDAS geometry files. More recent road geometry data collected in New Zealand however has not been produced in the same format, requiring conversion routines to be written to use it. A more flexible means of importing road data in a range of formats would assist here. Similarly, an ability to incorporate road data for planned new alignments would be desirable. Most new alignments are designed in terms of geometric elements (arcs, spirals, tangents, etc.) requiring further conversion routines to be developed to use this information in a format acceptable to TRARR.

Within TRARR’s ROAD files, running distances obtained from the original geometry data designate positions along the highway. For state highways with a series of Reference Stations (RS) and Route Positions (RP), correlating these running distances with RS/RP can be difficult (one way is to manually add comments to the ROAD file). It is further complicated by the fact that TRARR’s OBS (observations) files use a completely different running distance system to locate observation points. A means of bringing these different location systems together would be desirable.

For project evaluation purposes in New Zealand, the resulting speed and travel time data from TRARR must be translated into appropriate road user costs for different project options and traffic volumes. Without third-party utilities this can be a tedious exercise. It is accepted however that this would be a similar problem with most models unless developed specifically for Transfund.

Finally there is a distinct lack of practical documentation for running TRARR applications in New Zealand. The manual that comes with TRARR 4.0 (Shepherd 1994) is very slim and refers users back to the TRARR 3.2 manual (Hoban et al. 1991) for further details. While these documents provide very detailed information regarding the workings of the model, they do not address how to apply it practically in typical roading projects. For example, information on what to collect as suitable field data, how to calibrate the model, what to use as appropriate parameters for model runs, and how to apply the results to derive project benefits are not documented. It is left to modellers to develop this experience themselves by trial and error if they do not have the benefit of other experienced users to guide them.

5.2 Alternative Two-Lane Simulation Packages and Tools

As discussed in Section 5.1, a number of deficiencies within TRARR limit its ongoing usefulness as tool for evaluation in New Zealand. It will be necessary to consider possible alternatives in the future although, on the surface, there do not appear to be many.

5.2.1 TWOPAS

The US software package TWOPAS (St John & Harwood 1986) is an alternative tool that appears most worth investigating. TWOPAS was first developed in the mid-1970s by MidWest Research Institute and others for the Federal Highways Administration (FHWA). The operational analysis procedures for two-lane highways in the 1985 Highway Capacity Manual (HCM) are based on an early version of the TWOPAS model (Harwood et al. 1999). TWOPAS was revised most recently in 1998 (Leiman et al. 1998) to serve as the basis for updating the HCM analysis procedures in the 2000 version of the manual (TRB 2000). Some further work is now continuing to improve TWOPAS even further, and an updated version is planned to be available in early 2002. This ongoing development of TWOPAS makes it an attractive proposition to consider when compared with TRARR.

Like TRARR, TWOPAS is a microscopic computer simulation model of traffic on two-lane highways that simulates the operation of each individual vehicle on the roadway. The operation of each vehicle as it advances along the road is influenced by the characteristics of the vehicle and its driver, by the geometrics of the roadway, and by the surrounding traffic situation. The simulation contains a comprehensive overtaking sub-model allowing for both sight distance and opposing-vehicle-restricted opportunities, and for single and multiple overtakings. The decision logic is based on the sets of acceptability versus gap size and leader speed relations. The following features are found in TWOPAS:

- Three general vehicle types - passenger cars, recreational vehicles, and trucks.
- Roadway geometrics are specified as elements by the user in the input data including horizontal curves, grades, vertical curves, sight distance, passing lanes, climbing lanes, and short four-lane sections.
- Traffic controls are specified by the user, particularly passing and no-passing zones marked on the roadway.
- Traffic streams enter at each end of the simulated roadway generated in response to user-specified flow rate, traffic mix, and percent of traffic platooned.
- Variations in driver performance and preferences are based on field data.
- Driver speed choices in unimpeded traffic are based on user-specified distribution of driver-desired speeds.
- Driver-speed choices in impeded traffic are based on a car-following model that simulates driver preferences for following distances (headways), based on relative leader/follower speeds, driver-desired speeds, and desire to pass the leader.

- Driver decisions concerning initiating passing manoeuvres in the opposing lane, continuing/aborting passing manoeuvres, and returning to normal lane, are based on field data.
- Driver decisions concerning behaviour in passing/climbing/four-lane sections, including lane choice at beginning of added lane, lane changing/passing within added lanes and at lane drops (i.e. lane reductions), are based on field data.
- Processing of traffic and updating of vehicle speeds, accelerations, and position is done at intervals of one second of simulated time.

Recently TWOPAS has been updated to incorporate changes in driver and vehicle characteristics and to allow users to adjust for the effect on traffic operations of narrow lanes or shoulders.

While TWOPAS appears to be a promising alternative for TRARR, work is needed to confirm its validity and practicality for use here. Some issues, like its lack of direct compatibility with RGDAS road geometry data, need to be considered and solutions identified. If both TRARR and TWOPAS have irreconcilable problems in their long-term use New Zealand, then alternative solutions need to be sought.

5.2.2 Other Modelling Software

Both TRARR and TWOPAS are specifically designed for modelling rural highway links, and little else in the world has been currently developed for this application.

HUTSIM from Finland's VTT (McLean 1989) is one other recent example, however little documentation (particularly in English) has been sourced to date. It may be that a solution outside of traditional rural road modelling needs to be looked at and possibly adapted.

Much of the development of traffic models in recent times has focused on (often complex) urban and freeway networks. In the past, computing power has limited the ability to model these networks using micro-simulation, but now a number of software tools have been developed to tackle this. One such tool is PARAMICS, produced by SIAS (Scotland). PARAMICS simulates the individual components of traffic flow and congestion, and presents its output as a real-time visual display for traffic management and road network design (SIAS 1999). It has been developed over a period of ten years by UK traffic and transportation engineers. Although primarily aimed at urban networks and inter-urban motorways (where congestion effects tend to dictate vehicle speeds), it appears possible to apply PARAMICS or a similar model to a typical New Zealand rural highway, where geometric constraints may dictate.

Similar network micro-simulation packages such as AIMSUN (TSS, Spain) and DRACULA (ITS Leeds and WS Atkins, UK) have also been developed. Given the increasing use of these models in New Zealand for other applications, the relative merits of also applying them to rural highways are worth exploring.

Another possibility is to incorporate road-modelling features into existing road design packages. Programs such as MX-ROAD by Infracore can already produce sophisticated designs for realignments. Therefore, assessment of these designs in terms of road user benefits and costs may be more practical to build in, as this would enable more immediate feedback on the most optimal designs. This approach is touched on further later in the discussion on IHSDM (see Section 5.3.2).

5.3 Assessment of Relative Merits of Modelling Tools

In the past some comparisons of the capabilities and features of the TRARR and TWOPAS models have been undertaken with mixed results (Botha et al. 1993, Staba et al. 1991). Their capabilities and features were found to be comparable in many respects, with preference often based on local needs. For example, those with geometry data available in terms of elements found TWOPAS far more practical for their needs. Conversely, some found TRARR's simpler outputs to be more useable than TWOPAS's rather extensive results. It is significant however that, with suitably calibrated models, both models were found to give sufficiently adequate results.

Given the more recent developments of both models, it would be instructive to compare both again in a New Zealand setting. Possibly in a future Transit NZ state highway project, funds could be made available to test project options using both models. As discussed in Section 5.2, comparison with the more recent network micro-simulation models would also be of significant interest.

The level of detail involved in such simulation models may indeed provide little additional information than what could be provided from a simpler form of model. Generic simulation packages, such as SIMSCRIPT for example, could be used to develop a simplified vehicle interaction model that provides a sufficiently adequate level of accuracy. The feasibility of this approach is worth investigating at least.

5.3.1 Prediction of Safety Benefits

One of the problems with the simulation models described above is that they are designed specifically for evaluation of efficiency issues, e.g. travel-time savings and reductions in time spent following. Project evaluation of rural improvements in New Zealand invariably also requires an assessment of safety benefits, which currently has to be done separately from the above analysis. It would be preferable to incorporate both safety and efficiency analyses in the same process for testing and evaluating proposed roading improvements.

Most of New Zealand's existing crash analysis procedures are static "crash rate" models that relate typical crash rates and environmental modifying factors to actual traffic volumes. Another potential approach is to use micro-simulation to analyse driver/vehicle behaviour and identify what situations put road users at more risk than others. Because of the relatively rare nature of crashes, it is not expected that these models would necessarily simulate a vehicle crashing. Rather, proxies for unsafe behaviour can be used to assess likely crash rates, e.g. the number of aborted overtaking manoeuvres, or the number of vehicles exceeding the safe curve speed. A

similar approach was attempted by Kaub (1992) using a model based on an older simulation model ROADSIM. Safety benefits were estimated using a statistical probability conflict opportunity/cost model.

With a valid underlying rural simulation model, it would be desirable to use the same tool to check for speed environment consistency along the highway. This would help to identify the likely crash risk of the proposed new alignment or cross-section. Similarly the relative highway capacity levels of service provided along the road could also be assessed. Such tools would also have applications for strategic studies of longer lengths of highways to prioritise areas of most concern.

5.3.2 Interactive Highway Safety Design Model

A major project of relevant interest here is the Interactive Highway Safety Design Model (IHSDM), being carried out by the US Federal Highways Agency (FHWA 2001). IHSDM will be a suite of evaluation tools for assessing the safety impacts of geometric design decisions. IHSDM's evaluation capabilities will help planners and designers maximise the safety benefits of highway projects within the constraints of cost, environmental and other considerations. The initial development efforts are restricted to two-lane rural highways, with release of the full model for these highways scheduled for 2002. Further information is available on the FHWA website (FHWA 2001).

IHSDM consists of several analysis modules:

- Crash Prediction Module, to estimate the number and severity of crashes on specified roadway segments.
- Design Consistency Module, to provide information on the extent to which a roadway design conforms to drivers' expectations (especially speed profiles).
- Driver/Vehicle Module, consisting of a Driver Performance Model linked to a Vehicle Dynamics Model. These will estimate drivers' speed and path choice along a roadway and subsequent measures including lateral acceleration, friction demand, and rolling moment.
- Intersection Diagnostic Review Module, which will use an expert system approach to evaluate intersection design alternatives, identify geometric elements that may impact safety, and suggest countermeasures.
- Policy Review Module, to verify compliance with highway design policies.
- Traffic Analysis Module, using traffic simulation models to estimate the operational effects of road designs under current and projected traffic flows, e.g. travel times, vehicle interactions (this module is based on the TWOPAS model).

A considerable research effort has gone into the specification of the various modules. Although the finished modules may not be directly applicable to New Zealand conditions, the underlying research is still expected to be of significant value here. Therefore an investigation of the capabilities of IHSDM in a New Zealand context is warranted.

5.4 Discussion

Previous research in New Zealand has focused on small elements of the overall rural roading picture. Bennett (1994), for example, provided a comprehensive review and survey of free speeds as a function of road alignment; however no consideration was made of the effects of vehicle interaction. Research by Koorey et al. (1999) and this current report has investigated how passing opportunities, such as passing lanes and SVBs, can provide both efficiency and safety benefits. Cenek et al. (1997) meanwhile investigated the relationship between crashes and road geometry. There is a need to consider the overall impacts of changes to road alignments and cross-sections for both safety and efficiency in a unified manner.

To assist in prioritising future upgrading work for funding, detailed simulation modelling is required to firstly identify sections requiring improvement and secondly to determine the likely effect of improvement projects. There is currently little guidance for analysts on the most suitable simulation tools to use, and how to use them appropriately for various projects. Although more simplified procedures are being introduced to make things easier for both analysts and Transfund (e.g. passing lane procedures), there is still a need to validate such procedures using more robust analyses, before they are put to general use. Modelling is likely to be able to continue to provide that validation.

With the introduction of more simplified approaches, the use of modelling may move away from common use in specific project work. The level of accuracy gained from a simulation model may not justify the additional effort probably required to achieve it. Therefore the development of more sophisticated future models may be more for policy and research work rather than for general use by practitioners. The exception may be for very large construction projects, where accuracy of results is important before committing to a significant amount of funding.

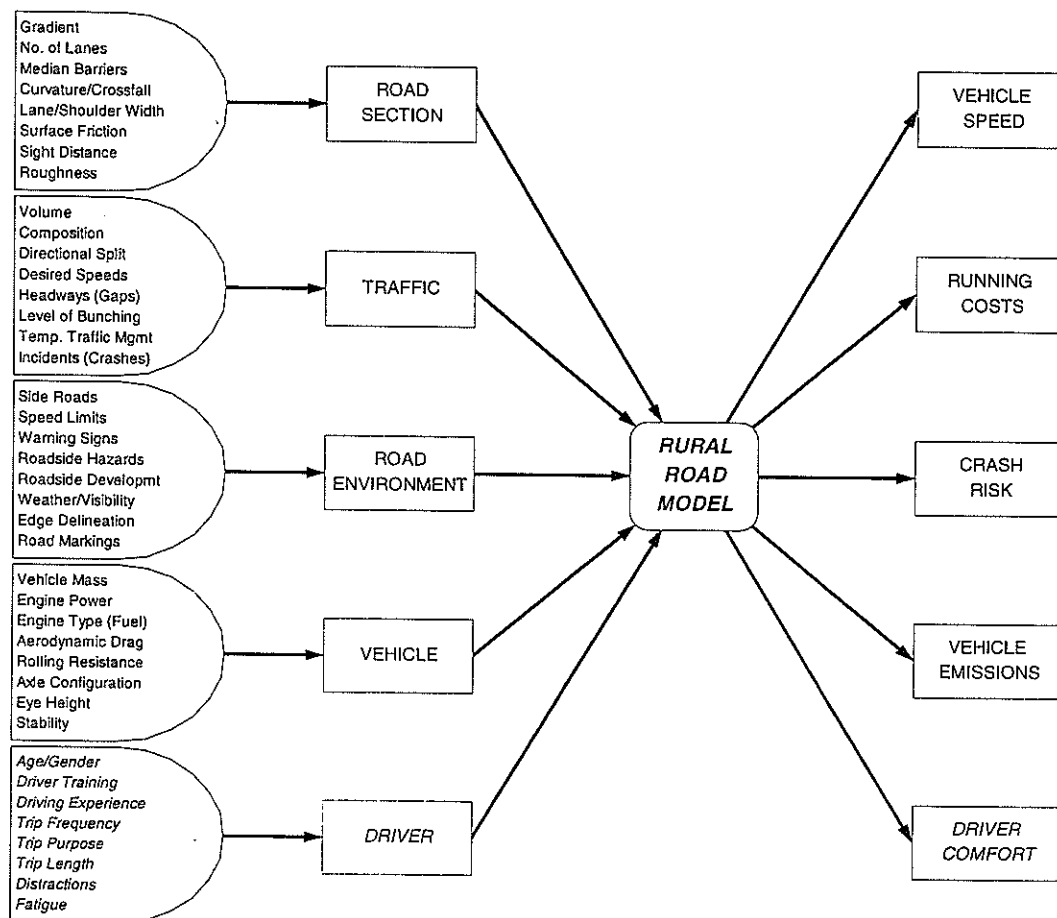
With the growing use of Intelligent Transport Systems (ITS) in New Zealand, simulation models may also provide a way to better model their effects (e.g. variable messages). This has applications for all high-speed highways, including rural roads/expressways and urban motorways.

5.5 A Framework for Rural Simulation Modelling in New Zealand

Figure 5.2 provides an overview of the requirements that an ideal rural simulation model needs to take account of. The various inputs on the left-hand side, categorised for clarity, interact in various ways to produce the outputs on the right-hand side.

This framework provides an over-arching basis for development of a model that provides all the required information for project evaluation in New Zealand, using as wide a set of information as possible. Comparison with existing models such as TRARR show that it fails to provide all the listed outputs, or to take account of all of the listed inputs.

Figure 5.2 A suggested framework for rural road modelling.



The diagram does not indicate the relative importance of each input, and maybe some of them can be ignored without significant effect on the final results (or at least for some outputs). It is likely also that some further inputs that have not been listed are equally as valid. In practical terms, reduction of the input data (or certainly the accuracy of some of it) may be necessary to minimise the costs in obtaining the data. There is little point in an improved model if the required effort to populate it and calibrate it is excessive.

The last input category, the driver, is not likely to be a direct input to a practical model for users. Rather, a representative series of behaviours is likely to be inferred based on observed actions on the road.

While some of the relationships between various inputs and outputs are well established, further research may be required to identify other relationships. This framework provides an initial starting point to help identify these “gaps” in knowledge.

As well as having the underlying theory sufficiently correct, a practical model for use in New Zealand and elsewhere ideally requires a number of other features to make it efficient to use. Some of these have been discussed above and include:

- Ability to create or import road data using a number of formats, particularly using either fixed-interval (e.g. 10m) or element-based geometry data.
- Easy editing of roadway information, particularly in relation to RCA route position systems.
- Ability to specify (or import) a range of time periods and traffic flow distributions/compositions.
- Ability to derive default or typical information where field data is not available (note: this approach needs to be treated with care).
- Customisation of outputs to produce desired information in specified format.
- Relating outputs to Transfund project evaluation benefits (discounted over 25 years).
- Integration of field data for more efficient calibration.
- Text-based input/output files for external editing if desired.
- Facility to change distributions used for some values (including using constant deterministic values).
- Ability to expand data requirements where desired (e.g. more vehicle types or longer road sections).
- A modern MS Windows or X-Windows graphical interface that uses standard features (buttons, drop-down lists, etc.).
- Ability to graphically display on plan and longitudinally the analysed routes and overlay various data sets (speeds, bunching, sight distances, etc.).
- Adequate documentation (both on-line help and written).

While a model with all of these features would greatly ease the workload for the user and assist with minimising data errors, it must be emphasised that this is of little consequence if the underlying model is not sufficiently robust enough.

6. Conclusions

This research has investigated a range of ways that passing opportunities are provided and evaluated. Although developments in this area have been continuing in New Zealand for some time now, it is evident that some things could still be introduced here, particularly with regards to policy for selecting and evaluating sites.

6.1 Driver Reaction to Changed No-overtaking Markings

Field surveys and subsequent desktop analysis of three short overtaking sites in New Zealand, together with literature review, revealed:

- Small amounts of overtaking (averaging up to 2.5% of all traffic) do occur at sites considered to be sub-standard in terms of horizontal sight distance (SD).
- The maximum potential amount of overtaking occurring is inversely proportional to the amount of traffic in the opposing direction.
- Changing no-overtaking markings to include horizontal curve criteria is estimated to at least double the amount of marking required.
- A change to horizontal curve no-overtaking would reduce overtaking rates by up to 50% (less on flat terrain), and increase time spent following by 20%-30%. However travel times would increase by no more than 9%, and less where traffic is greater.
- Likely changes in driver compliance with the introduction of more restrictive no-overtaking criteria do not have a significant effect on the predicted changes in travel times and overtaking rates.
- The predicted reduction in “dangerous” overtaking manoeuvres will be somewhat offset by the increase in driver frustration. However a net safety benefit is expected with the introduction of more restrictive no-overtaking criteria.
- Allowing no-overtaking lines to be marked on straight approaches to isolated horizontal curves, to inform drivers when suitable overtaking lengths are no longer available, may provide a suitable compromise between the existing vertical-only SD criteria and a full combined horizontal/vertical SD criteria.

6.2 Field Calibration of Simplified Procedures

Field surveys, literature review, and various TRARR and simplified procedure analyses on passing lane situations showed:

- The recent updating of US Highway Capacity Manual procedures provide a fairly straightforward means of assessing the effects of a passing lane on vehicle bunching and travel speeds along a two-lane highway. It may be reasonable to use them to assess passing lanes in New Zealand, either at a project specific level or for strategic route studies.

- A revised Willingness-To-Pay (WTP) relationship has been derived that relates driver frustration to changes in Percent Time Spent Following (PTSF). This produces WTP values that vary with different passing demands.
- Accrued Passing Demand (APD) values have been understated because passing demand within vehicle queues has not been included in Transfind's simplified procedures. At high bunching levels, the true APD can be more than double that inferred from observed vehicle bunching.
- Passing Demand calculations should be reduced as the APD approaches the actual traffic volume. By applying a scaling factor and capping APD to no more than 90% of volume, more realistic passing demands can be determined.
- Maximum passing rates are generally proportional to traffic volumes, with maximum rates on passing lanes approximately double that on a two-lane highway with no sight distance or opposing traffic restrictions. At lower levels of passing demand however, observed rates are usually less.
- Even with the suggested changes, the simplified procedures still tend to over-estimate passing demand relative to supply on two-lane highway sections and supply relative to demand on passing lanes. What appears to be missing is some "equilibrium" value that a given road and traffic situation will gravitate towards.
- The relationship between APD and bunching is still not entirely certain, and maybe the simplified procedure should be re-presented in terms of %Bunching values instead.

6.3 Field Performance of Slow Vehicle Bays

Field surveys and subsequent desktop analysis of eight slow vehicle bay (SVB) sites in New Zealand, together with literature review, revealed:

- Unlike passing lanes, SVB use appears to be very dependent on the location and design of each site; a poorly placed or insufficiently long SVB can suffer low usage and provide little benefit.
- The use of SVBs by platoon leaders generally ranges between 30%-60%, a higher rate than that found in overseas studies. Use increased by more than 10% on average for platoons with more than one vehicle following.
- SVBs do not greatly reduce the proportion of bunched vehicles, particularly in winding alignments. Less than 10% (absolute) reductions in vehicles following were observed at all sites (and some increased), although generally between 10%-20% of all vehicles used an SVB. The short-term benefits however probably do provide some reduction in driver frustration.
- Trucks and recreational vehicles typically made up 70%-90% of all vehicles using SVBs, with trucks in particular being high users (30%-50% of all trucks). Some sites that looked more like passing lanes had higher car use.
- Evidently confusion by drivers exists over the use of some SVBs. This is seen in their relatively high use by vehicles when no one is following and in conflicts at SVB merges.

- The Borel-Tanner Distribution provided an excellent model for bunch sizes observed at SVBs. Using this model, drivers are more likely to find themselves following in a multi-vehicle queue, than following a single vehicle, when the overall percentage of following is greater than 23%. Therefore queues of two or more vehicles following play a significant part when considering the operation of SVBs.
- The current New Zealand guidelines for SVB lengths may be inappropriate, given the number of merge area conflicts and the likely number of multi-vehicle queues. An analysis of minimum required lengths to allow one or two vehicles to safely pass another vehicle without greatly impeding it showed that many high-speed situations require longer than the recommended 300m maximum length.
- For a short SVB with sufficient queuing, any travel time benefits gained by the passing vehicles may be negated by the delay placed on the overtaken vehicle.
- SVBs modelled by TRARR, using a modified PBAYS file, provide a realistic reduction in the proportion of platooned vehicles. However TRARR underestimates the actual travel-time savings. Re-modelling the SVB as a short passing lane is likely to over-estimate travel-time savings, and the true answer will be dependent on the likelihood of overtaken vehicles having to slow down or stop.
- A simplified formula has been developed that appears adequate for predicting bunching rates after an SVB, given initial on-site field surveys, and can be applied to the calculation of the frustration benefits from a proposed SVB.
- The safety benefits of SVBs are not entirely clear. A poorly designed merge area or unacceptably short SVB may cause a high level of serious conflicts. Downstream crash savings beyond the site may be very limited, but at the site savings are likely if the new SVB provides a safer alternative to previous overtaking attempts at that site.

6.4 Development of Framework for Detailed Modelling

Literature review and assessment of various highway simulation models identified a number of observations:

- Previous research in New Zealand has focused on isolated elements of the overall rural roading picture, such as speed prediction, passing opportunities, and crashes related to geometry. There is a need to consider, in a unified manner, the overall impacts of changes to road alignments and cross-sections, for both safety and efficiency.
- Although simplified procedures are helping to ease the effort required to evaluate rural improvements, simulation would still appear to provide more accurate information when evaluating realignments, multiple passing lanes, or continuous four-laning. Any move to other “simplified” methods, as is done for passing lanes, will also require research using simulation to confirm the validity of the methods for general use.

- TRARR has some problems when evaluating local scenarios. Some problems identified include its inability to handle side road flows, lack of consideration for speed zones and road widths, susceptibility when modelling highly congested situations, and a difficulty in using field data for calibration. Also more practical concerns are with the TRARR software, including its DOS-based interface, inflexibility when importing road data, its inability to relate to Transfund project evaluation requirements, and lack of practical documentation.
- TRARR now lacks ongoing support by its publishers, ARRB Transport Research. A similar US software tool, TWOPAS, which is currently undergoing major revision by the FHWA, holds some promise as a possible replacement. However, work is needed to confirm its validity and practicality for use in New Zealand.
- An alternative approach is to consider the feasibility of using more recent micro-simulation tools like PARAMICS or AIMSUN for rural modelling work. Although primarily aimed at urban networks and inter-urban motorways (where congestion effects tend to dictate vehicle speeds), these models appear possible to be applied to a typical New Zealand rural highway, where geometric constraints may dictate.
- A general framework has been developed in our research that provides an overview of what an ideal rural simulation model needs to take account of, in terms of how various road, traffic, vehicle and driver inputs interact to provide all the information required for project evaluation in New Zealand. While some of the relationships between various inputs and outputs are well established, the framework provides an initial starting point to help identify research on the “gaps” in knowledge.

7. Recommendations

The following items are recommended for further investigation or action.

7.1 Driver Reaction to Changed No-overtaking Markings

- Further overtaking surveys following implementation of revised no-overtaking marking criteria may be useful to assess the likelihood of driver compliance. Alternatively, the use of driving simulator studies would allow various marking options to be tested with some degree of safety first.
- Similar studies of specific site situations, such as opposing passing lanes, major intersections, and narrow bridges, also need to be investigated to assess the effect of no-overtaking markings on compliance and safety.
- The use of a speed-dependent measure for assessing no-overtaking areas, such as Intermediate Sight Distance (ISD), should be considered in New Zealand.
- No-overtaking lines should be marked on straight approaches to isolated horizontal curves, to inform drivers when suitable overtaking lengths are no longer available. A suggested criterion is to only mark sites where the available sight distance is between $0.5-1.0 \times \text{ISD}$.

7.2 Field Calibration of Simplified Procedures

- The revised US HCM procedures for passing lanes should be investigated further in New Zealand to confirm their appropriateness for evaluating passing lane projects.
- The revised driver frustration WTP procedure should be applied to passing lane projects in New Zealand.
- The suggested changes to Transfund's simplified passing lane procedures should be incorporated to improve the robustness of their model.
- Further testing should be considered before widening the scope of the simplified procedures to more complex situations such as multiple passing lanes.

7.3 Field Performance of Slow Vehicle Bays

- SVBs should be clearly marked distinctly from passing lanes, to prevent confusion of the two by drivers. The length of each site should also be considered when deciding whether they qualify as an SVB or a passing lane.
- Driver education should be carried out on the purpose and correct use of SVBs.
- To maintain the adequacy of the 300m maximum recommended length, SVBs should be located only where the mean traffic speed is less than about 60km/hr. Where traffic volumes are greater and longer queues are more likely, even lower mean traffic speeds are desired installing an SVB is considered.

- An updated version of TRARR's PBAYS file, that has been developed in this research, should be used for TRARR SVB modelling in New Zealand. For assessing travel time savings, a comparison should be made against modelling the site as a short passing lane instead, and a reasonable figure should be determined on the basis of minimum required passing lengths.
- Travel time savings achieved from SVBs should only be considered where the site does not cause undue delay to those vehicles being overtaken that are waiting to re-enter the traffic stream.
- An alternative simplified evaluation procedure should be used for SVBs based on these research findings, instead of the existing simplified passing lane procedures.

7.4 Development of Framework for Detailed Modelling

- The most recent versions of the TRARR and TWOPAS models should be compared in a range of New Zealand settings and scenarios with actual field data. A comparison with the more recent network micro-simulation models, such as PARAMICS and AIMSUN would also be of significant interest.
- Further work should incorporate safety assessments in rural simulation models used in New Zealand. Related to this, an investigation of the capabilities of US IHSDM project in a New Zealand context is warranted.
- An improved rural simulation model should either be adopted or developed, following the further investigation of the various alternatives.

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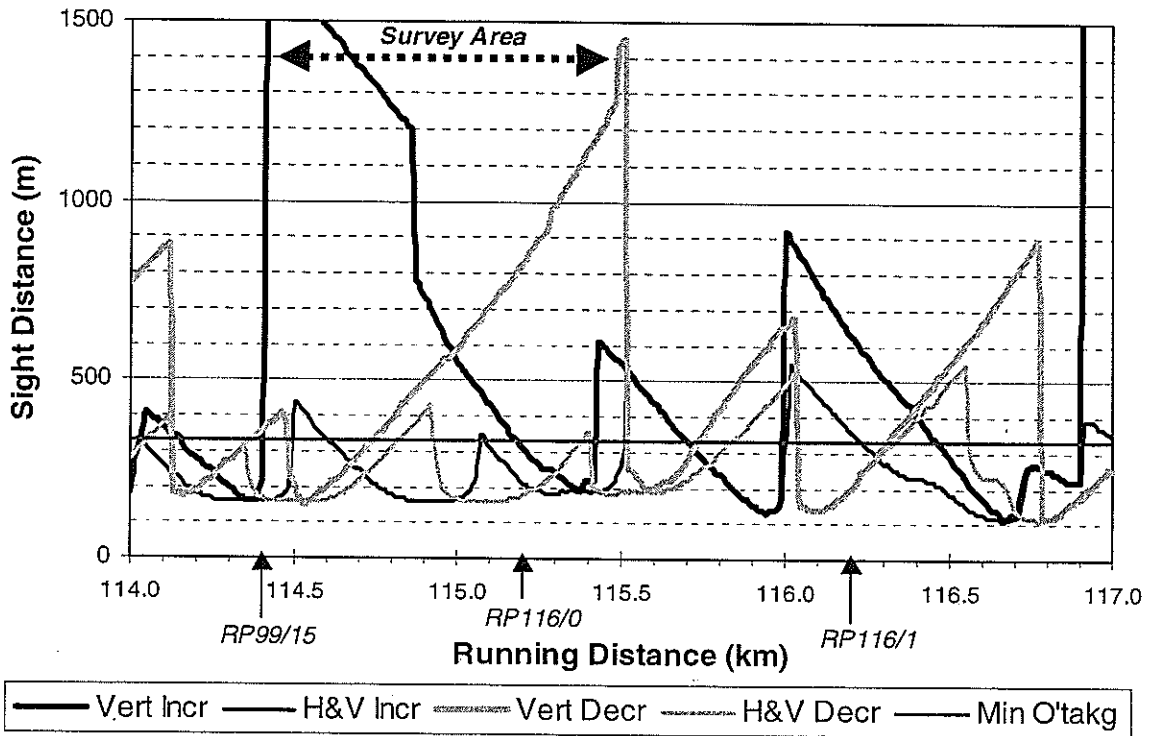
Appendices

A1 Original Overtaking Survey Sites

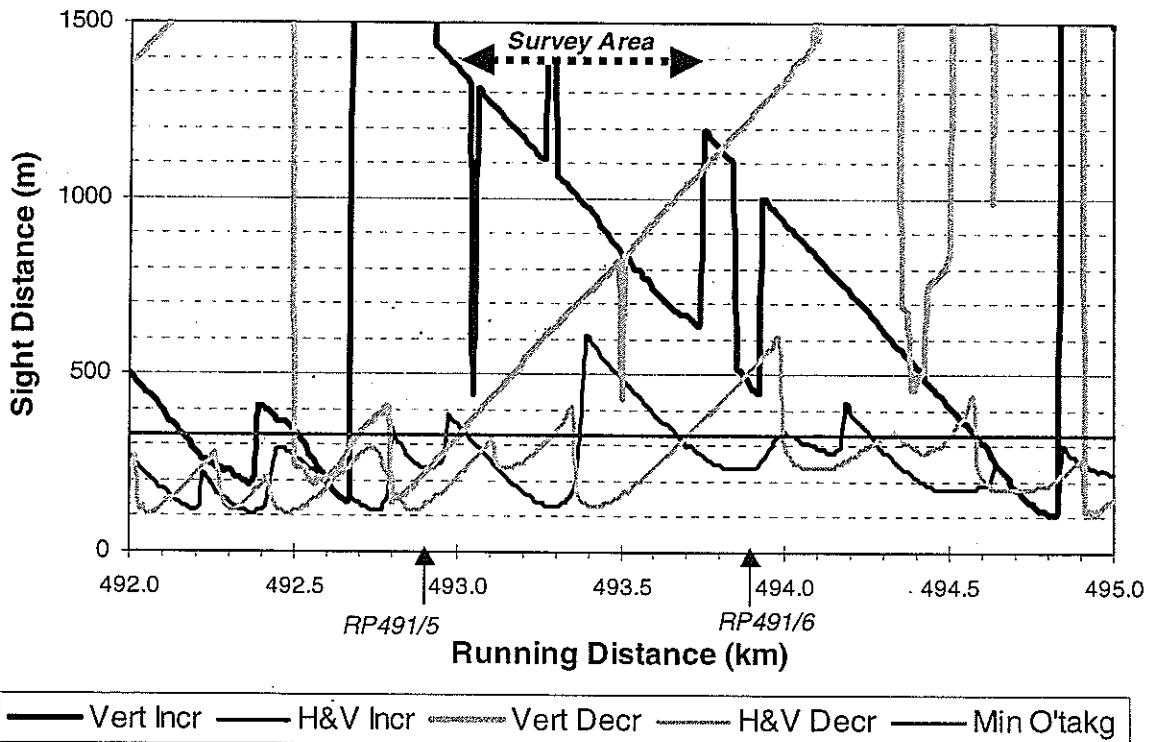
SH	RS	Start	End	Description	Otak'g Inj.Accs	AADT	Comments
1N	144	10.9	11.7	N of Whangarei	5	6000	Unsuitable, passing lane, speed limits
1N	165	5.4	6.3	S of Whangarei	4	11000	Unsuitable, no-o'taking lines, intersection
1N	173	3.3	4.5	Whangarei - Waipu	7	8000	Unsuitable, passing lane
1N	173	6.6	7.6	Whangarei - Waipu	5	8000	May be OK
1N	203	15.2	16.0	Waipu - SH12	4	6000	Unsuitable, intersection
1N	274	7.8	8.7	Warkworth - Orewa	5	13000	No details obtained
1N	274	12.4	13.4	Warkworth - Orewa	7	13000	No details obtained
1N	288	11.7	12.4	S of Orewa	4	20000	Unsuitable, high traffic
1N	303	7.4	8.4	Orewa - Albany	6	24000	Unsuitable, high traffic
1N	393	7.0	7.1	SH2/Pokeno - Huntly	4	12000	Not very suitable, tight curve
1N	412	10.8	11.7	SH2/Pokeno - Huntly	7	12000	Unsuitable, limited sight distance
1N	780	10.1	10.2	Mangaweka - Hunterville	10	5000	No details obtained
1N	903	6.4	6.9	SH57/Levin - Otaki	8	13000	Unsuitable, limited sight distance
1N	915	3.5	4.4	SH57/Levin - Otaki	8	13000	No details obtained
1N	942	8.5	9.6	Paraparaumu - Paekak	8	22000	Unsuitable, high traffic
1N	942	9.9	10.5	Paraparaumu - Paekak	5	22000	Unsuitable, high traffic
1N	953	10.1	11.2	Paekakariki - Porirua	10	21000	Unsuitable, high traffic, construction
2	116	0.0	1.0	Waihi - Katikati	5	8000	Probably OK
2	164	6.8	7.2	E of Mt Maunganui	4	13000	Unsuitable, no o'taking lines, intersection
2	164	8.1	8.9	E of Mt Maunganui	5	14000	Unsuitable, no o'taking lines, intersection
2	294	2.2	3.8	W of SH35/Opotiki	7	3000	May be OK
2	707	10.4	11.1	Waipawa - Waipukurau	4	7500	Probably unsuitable, limited visibility
2	931	1.6	3.4	Rimutaka Saddle	14	4500	May be suitable, short passing bays in vicinity
2	931	8.9	10.3	Rimutaka - Upper Hutt	10	5000	May be suitable, passing lane immediately after
2	946	0.3	0.5	Rimutaka - Upper Hutt	4	6000	Unsuitable, no o'taking lines, intersection
3	229	5.6	5.9	E of New Plymouth	5	11000	No details obtained
3	450	3.3	3.9	E of SH1/Sanson	4	7500	Unsuitable, construction
3	491	0.1	1.4	Manawatu Gorge	7	6000	May be suitable, difficult site access
3	491	2.0	3.8	Manawatu Gorge	9	6000	May be suitable, limited visibility
3	491	5.3	6.1	W of SH2/Woodville	5	6000	Suitable
4	223	7.4	8.4	N of Wanganui	4	2000	Unsuitable, reconstructed
5	8	0.0	3.0	E of SH28	10	4000	Unsuitable, no o'taking lines, intersection
5	8	5.6	6.0	E of SH28	6	4000	Unsuitable, intersection
6	80	18.7	19.3	Whangamoia - Nelson	5	2500	Incorrect RP
6	201	7.7	7.9	N of SH63	5	1500	Unsuitable, construction
16	19	0.9	1.9	Waitakere - Kumeu	6	22000	Unsuitable, high traffic
18	0	4.7	5.3	North Shore - Whenuapai	8	21000	Unsuitable, high traffic
57	0	15.5	18.1	SH1/Levin - Shannon	12	6000	Unsuitable, construction
58	0	11.4	12.3	Pauatahanui - Porirua	8	6500	Suitable
58	0	13.7	14.5	Pauatahanui - Porirua	6	12000	May be suitable, visibility across harbour
60	33	3.6	4.4	N of Motueka	6	6000	Unsuitable, turning traffic

A2 Sight Distance Plots of Selected Survey Sites

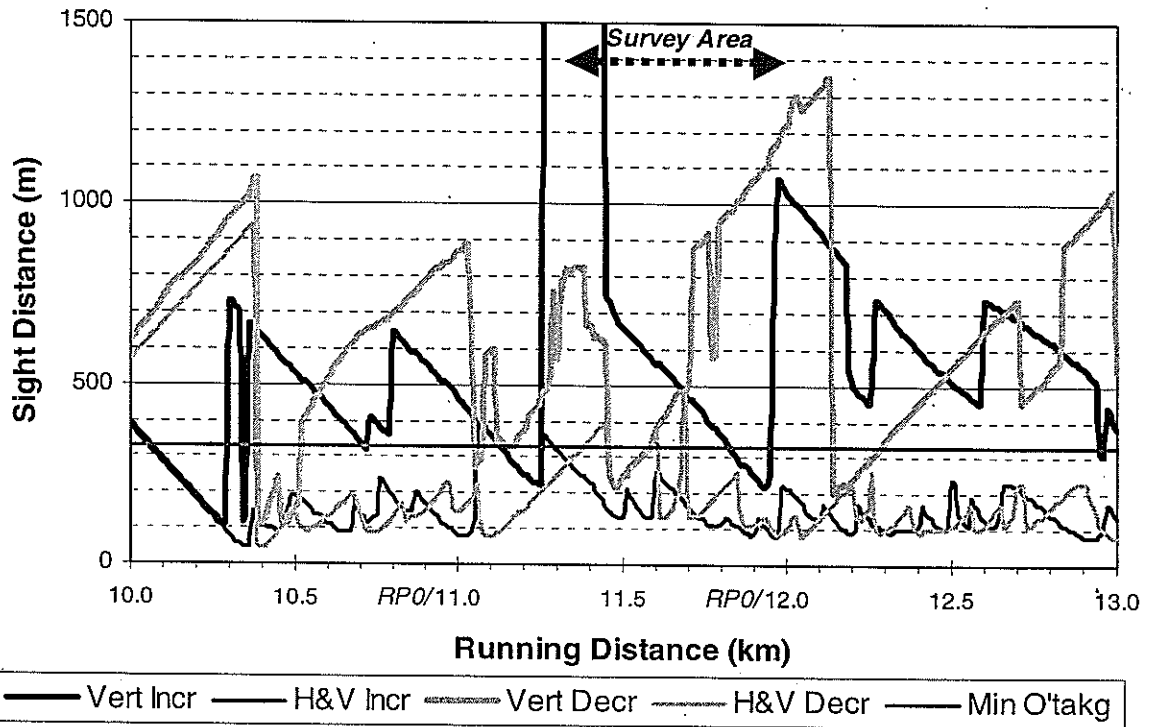
SH2/116 Waihi – Katikati



SH3/491 West of Woodville



SH58/0 Pauatahanui – Porirua



A3 Slow Vehicle Bay Sites Identified in New Zealand

Dir/n direction: Incr – increases; Decr – decreases from RS/RP
 Grade in % slope; Length in metres; AADT in veh/day; HCV in %

SH	RS/RP	Dir/n	Section	Name	Grade	Length	AADT	%HCV	Comments
IN	16/14.3	Incr	Sth of Kaitaia	Mangamuka	+9.5	90	900	12	
IN	32/2.5	Decr	Sth of Kaitaia	Mangamuka	+5.0	110	900	10	
IN	47/11.4	Decr	Sth of Kaitaia	Umawera	+6.0	300	1400	10	
IN	113/10.0	Decr	Sth of Kawakawa	Towai	+3.0	90	5200	17	
IN	203/15.5	Decr	Waipu - Wellsford		+0.0	100	7100	8	
IS	601/8.1	Decr	Oamaru - Dunedin	Pigeon Flat	-8.5	900	3900	11	Original markings retained
IS	667/14.2	Incr	Oamaru - Dunedin	Kilmog	-10.0	450	3900	11	Original markings retained
IS	683/16.2	Incr	Oamaru - Dunedin	Thomsons Curve	-6.0		3900	11	
IS	774/0.7	Incr	Milton - Balclutha		+3.5		4200	14	From RAMM signs inventory
2	361/3.3	Decr	Opotiki - Gisborne	11.02km nth of Matawai	+8.0	180	800	10	
2	375/5.8	Decr	Opotiki - Gisborne	5.21km Sth of Matawai	+5.0	350	800	10	
2	390/2.8	Decr	Opotiki - Gisborne	17.42km Sth of Matawai	+1.0	231	800	10	
2	390/5.4	Incr	Opotiki - Gisborne	20km Sth of Matawai	+7.0	125	800	10	
2	390/5.9	Decr	Opotiki - Gisborne	20.54km Sth of Matawai	-3.8	210	800	10	
2	390/8.1	Incr	Opotiki - Gisborne		-5.0		1400	16	From RAMM signs inventory
2	390/10.1	Decr	Opotiki - Gisborne	24.69km Sth of Matawai	+5.5	140	1400	10	
2	544/7.8	Decr	Wairoa - Napier		+8.5	0	1300	15	From RAMM signs inventory
2	562/2.1	Decr	Wairoa - Napier	Mohaka Ascent	+9.0	250	1400	13	
2	562/4.7	Decr	Wairoa - Napier	Nth Raupunga Ascent	+10.0	350	1400	13	
2	562/13.5	Decr	Wairoa - Napier	Bluegum(Nth Chimney Creek)	+7.0	100	1400	13	
2	577/8.1	Decr	Wairoa - Napier		+9.0	0	1400	17	From RAMM signs inventory
2	577/13.4	Decr	Wairoa - Napier	Waikere	+8.0	300	1400	13	
2	592/6.2	Incr	Wairoa - Napier	Matahorua Gorge descent	-6.0	60	1400	13	
2	592/6.2	Decr	Wairoa - Napier	Matahorua Gorge descent	+7.5	200	1400	13	
2	592/8.0	Incr	Wairoa - Napier	Matahorua Gorge ascent	+8.0	180	1400	13	

ASSESSING PASSING OPPORTUNITIES – STAGE 3

SH	RS/RP	Dirn	Section	Name	Grade	Length	AADT	%HCV	Comments
2	592/11.8	Decr	Wairoa - Napier	Melrose	+1.0	120	1400	13	
2	608/5.1	Incr	Wairoa - Napier	Waikoau Hill	-7.0	100	1600	18	
2	608/11.8	Incr	Wairoa - Napier	Darkys Spur	-7.0	550	1600	13	
2	608/11.8	Decr	Wairoa - Napier	Darkys Spur	+6.0	550	1600	13	
2	608/15.9	Incr	Wairoa - Napier	Kareara Bridge	-3.5	80	1600	13	
2	921/0.7	Incr	Featherston-Upper Hutt		+6.0	270	4200	5	Now marked as passing lanes
2	921/3.4	Incr	Featherston-Upper Hutt		+5.0	340	4200	5	Now marked as passing lanes
2	921/5.6	Incr	Featherston-Upper Hutt		+8.5	90	4200	5	Now marked as passing lanes
2	921/8.1	Incr	Featherston-Upper Hutt		+9.0	170	4200	5	Now marked as passing lanes
2	921/9.5	Incr	Featherston-Upper Hutt		+6.0	190	4200	5	Now marked as passing lanes
3	176/5.5	Decr	Mokau-Waitara		+10.0		2000	9	From RAMM signs inventory
4	188/6.5	Incr	Raetihi-Wanganui		+0.7	200	1000	12	
5	111/9.7	Incr	Rotorua - Taupo	Palmer's Mill South	+5.0	250	3200	12	
5	233/4.2	Decr	Taupo - Napier	Mistletoe	+0.0	200	2300	14	
6	956/1.3	Incr	Cromwell - Queenstown		+7.0		1900	9	From RAMM signs inventory
7	152/4.6	Incr	Springs Jn - Reefton	Rahu Saddle	+4.5	250	1900	20	
7	152/5.9	Incr	Springs Jn - Reefton	Rahu Saddle	+7.0	90	1900	20	
7	152/11.0	Decr	Springs Jn - Reefton	Rahu Saddle	+4.5	260	1900	20	
8	343/3.1	Incr	Alexandra - Milton		+4.0		1600	9	From RAMM signs inventory
8	444/4.1	Incr	Alexandra - Milton	Rhodes Road	+0.0	640	1500	9	
8	444/4.7	Decr	Alexandra - Milton		+4.0		1500	9	From RAMM signs inventory
8	444/4.9	Decr	Alexandra - Milton		+3.0		1500	9	From RAMM signs inventory
10	0/11.5	Incr	Pakaraka - Mangonui		+9.0	110	2800	10	
10	95/4.1	Incr	Pakaraka - Mangonui		+9.5	70	2200	9	
14	30/10.3	Decr	Whangarei - Dargaville		+5.0	100	1500	9	From RAMM signs inventory
29	21/7.6	Decr	Tirau - Tauranga	Kaimai Range	-8.0		6000	17	
29	21/14.7	Decr	Tirau - Tauranga	Cannonball Deer Farm	-9.0	150	5850	15	
29	21/15.7	Decr	Tirau - Tauranga	Old Kaimai Rd	-5.0	200	5850	15	

SH	RS/RP	Dirn	Section	Name	Grade	Length	AADT	%HCY	Comments
29	21/19.0	Decr	Tirau - Tauranga	Kaimai Deer Farm	-8.0	200	5850	15	
29	42/0.5	Incr	Tirau - Tauranga	Kaimai Range	-6.0		5850	15	
29	42/1.8	Incr	Tirau - Tauranga	Kaimai Range	-12.0		5850	15	
29	42/2.3	Incr	Tirau - Tauranga	Kaimai Range	-9.0		5850	15	
29	42/3.1	Incr	Tirau - Tauranga	Kaimai Range	-8.0		5850	15	
29	42/3.7	Incr	Tirau - Tauranga	Kaimai Range	-9.0		5850	15	
35	213/5.1	Decr	Te Araroa - Gisborne	17.15km Sth of Ruatoria	+11.0	230	800	12	
35	225/5.2	Decr	Te Araroa - Gisborne	3.58km Sth of Te Puia	+11.0	120	800	11	
35	274/8.4	Decr	Te Araroa - Gisborne	8.65km Sth of Tolaga Bay	+8.0	180	1000	11	
35	289/5.2	Incr	Te Araroa - Gisborne	20.73km Sth of Tolaga Bay	+3.0	220	1200	11	
35	289/7.2	Decr	Te Araroa - Gisborne	22.74km Sth of Tolaga Bay	+6.0	280	1200	11	
43	0/7.0	Decr	Stratford - Taumarunui		-4.0	200	1100	9	
43	32/13.0	Incr	Stratford - Taumarunui		+5.0	50	250	9	
43	57/0.3	Decr	Stratford - Taumarunui		+0.0		250	9	From RAMM signs inventory
60	42/4.2	Incr	Motueka - Takaka		+9.5		1200	9	From RAMM signs inventory
60	42/7.1	Incr	Motueka - Takaka		+6.0		1200	9	From RAMM signs inventory
60	56/9.6	Decr	Motueka - Takaka		+11.0		1100	9	From RAMM signs inventory
75	52/5.6	Incr	Christchurch - Akaroa	Hilltop, Akaroa Road	+6.0	150	1600		
75	61/4.4	Decr	Christchurch - Akaroa	Hilltop, Akaroa Road	+12.0	250	1500		
6A	0/6.1	Decr	Frankton - Queenstown		+8.0		11000	7	

A4 Slow Vehicle Bay Observation Survey Instructions

(Survey Location to be specified separately)

These surveys aim to collect some observation-based information at slow vehicle bay (SVB) sites, using **two surveyors**:

- One surveyor collects % **Following** data for traffic (i.e. the proportion of following vehicles) **immediately prior** to the SVB. They also record the **level of use** of the SVB, i.e. who uses which lane.
- Another surveyor collects % **Following** data for traffic, **some distance after** the end of the SVB (suggest at about 500m). This is to assess the effect that the SVB has had.

General Notes:

- All data is collected in the **direction of the SVB only**, i.e. ignore the opposing direction.
- Use the attached form to record vehicles (see sample sheet for example of filling in). Record **all** vehicles that arrive, one per line (up to three columns of 40 vehicles per sheet). When surveying beyond the SVB, the "Use SVB?" column doesn't need to be filled in (cross out the appropriate choice for "Prior/After SVB" to indicate which site you are surveying).
- For each vehicle, record the vehicle type, whether it is following, and (for the "prior" surveyor) whether it uses the SVB. **Record clearly** using ticks or crosses.
- "Free" (i.e. on their own) or "Lead" (i.e. leading a queue) vehicles will not require a tick in the "Follow?" column, but all other "Following" vehicles will.
- Use your judgement to decide if a vehicle is free or following. As a guide, there should be **no more than 4-5 seconds** between them (try counting "one-thousand-and-one, one-thousand-and-two..." etc).
- Vehicles should be **classified** into "**Car**" (passenger cars, stn wagons, vans, utes), "**Rec**" (recreation vehicles, such as towing vehicles), and "**Trk**" (MCVs, HCVs, buses).
- To enable the data to be compared with volume, a new sheet should be started **every half hour**. Ideally both surveyors should do this simultaneously, so that the same vehicles are recorded in the same time period - consider using walkie-talkies or similar to co-ordinate this.
- A **minimum of four hours** of surveying (= 8 sheets each) should be carried out (surveyors may like to swap over after two hours). Breaks during recording are allowed, so long as exactly the same vehicles are recorded by both surveyors.
- Every attempt should be made to be as **inconspicuous** as possible (hidden off the road is ideal). Drivers may alter their behaviour if they are aware of your presence.
- Please **take some photos** of the site from both directions, to show the layout/extent of the SVB and surrounding environment.

Return all completed survey forms/photos to:

Glen Koorey, Opus Central Labs
PO Box 30-845, Lower Hutt
Ph: 04 587 0619 (x8619)

Any questions/problems, please contact Glen. **Thank You**

A5 Updated TRARR PBAYS File

The key changes are in the LQLPB row. Note that the NPB (Number Passing Bays) parameter and VPBP row should be adjusted to include details of the specific SVB(s) being modelled.

FILE PBAYS

THE FOLLOWING PARAMETERS ARE USED ONLY FOR PASSING BAYS (PBAYS)

NPB	DFOLPB	TFOLPB	DSTLPB	TSTLPB
2	20.	4.	100.	1.

QUEUE BEHIND SUCH THAT USE PASSING BAY (0 FOR NEVER) BY VEHICLE TYPE (LQLPB)
1 1 1 1 1 1 1 1 0 0 0 1 1 2 1 0 0 0

DIRECTIONS OF TRAVEL FOR PASSING BAYS

1 2

POSITIONS OF PASSING BAYS (DISTANCES FROM START OF SIMULATED SEGMENT) (VPBP)
150. 1200.

PASSING BAY DATA: TO BE READ ON PBAYS.

1. TRARR DOES NOT CONSIDER THE LENGTH OF THE PASSING BAY, BUT ONLY ITS EXISTENCE AT A PARTICULAR LOCATION
2. A VEHICLE ENTERS THE PASSING BAY IF THE BAY IS EMPTY AND THERE IS A QUEUE OF VEHICLE SET BY THE PRECEDING PARAMETERS
3. A VEHICLE LEAVES THE BAY WHEN THERE IS NO VEHICLE BEHIND IT FOR DSTLPB M PLUS TSTLPB S
4. NOTE: TRARR WILL INSERT OVERTAKING BARRIER LINES ALONGSIDE A PASSING BAY IF THESE DO NOT ALREADY EXIST

#EOF

