Operating characteristics and economic evaluation of 2+1 lanes with or without intelligent transport systems assisted merging

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An important note for the reader

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Abbreviations and acronyms

2+1	a section of carriageway with an alternating passing lane configuration in each direction of travel
AADT	annual average daily traffic
ANPR	automatic number plate recognition
Austroads	Association of Australian and NZ Road Transport and Traffic authorities
BCR	benefit–cost ratio
EEM	*Economic evaluation manual*
EL	effective downstream length (downstream)
GPS	global positioning system
HCV	heavy commercial vehicle
HV	heavy vehicle
ITS	intelligent transport systems
LI	lead-in section to passing lane
LOS	level of service
ML	modelled length of road
MoTSaM	*Manual of traffic signs and markings*
PL	passing lane
Safe System	internationally recognised system to reduce death and serious injury crashes
SH1/SH2	State Highway 1/State Highway 2
SHGDM	*State highway geometric design manual*
SP	passing lane spacing
SVB	slow vehicle bay
TMS	traffic monitoring system
TT	travel time
VMS	variable message signs
VOC	vehicle operating cost
vpd	vehicles per day
vph	vehicles per hour
WRB	wire rope barrier
WS2+1	wide single 2+1 (see 2+1)

Contents

Executive summary

The rural New Zealand strategic road network predominantly consists of two-lane two-way roads. This component of the state highway network has an important strategic function linking urban areas, tourist destinations, playing a key role in the movement of freight and carrying 30% of New Zealand's total annual road travel (vehicle kilometres travelled). Currently the large proportion of this network (around 73% in terms of length) carries lower traffic volumes. In the future a much higher proportion of the network will carry moderate to higher volumes: 5,000 to over 25,000 annual average daily traffic (AADT) two-way flow.

A number of countries have adopted a three-lane configuration referred to as a '2+1 roadway' at moderate to higher volumes to bridge the gap between two-lane and four-lane roads. A 2+1 roadway has alternating passing lanes by direction, often utilising a continuous three-lane cross section for some route segments. Despite its lack of New Zealand implementation, the 2+1 roadway is recognised within the New Zealand passing lane strategy as an intermediate step to four-laning.

The research project is applicable to New Zealand's state highways and has established economic optimisation principles and influential factors which affect the characteristics and operation of passing lane treatments in series at moderate to higher volumes across the various terrains within New Zealand. The project's review of international experience of passing lane and 2+1 safety and operational performance has enabled the development of a 2+1 roadway design which is potentially suitable for New Zealand. Other current research is addressing related matters which when complete will provide a broad understanding of design factors in the New Zealand context.

Extensive microsimulation traffic modelling has been undertaken to establish the optimum passing lane length and spacing combinations which deliver the maximum benefits for combinations of terrain, AADT, traffic growth forecasts and the percentage of heavy commercial vehicles.

In high-volume environments, the merges of passing lanes and 2+1 roadways can create delays which affect traffic flow, reduce traffic speeds and increase the risk of collisions. A range of merge concepts assisted by intelligent transport systems (ITS) was investigated to establish if the deployment of technology could enable vehicles to merge at higher volumes and thereby extend the service life of passing lanes and 2+1 roadways. The ITS-assisted merge concepts considered the action of managing headways, speeds, merge 'location' and lane use. The most efficient concept involved the introduction of speed restrictions within the passing lane when traffic volume increased beyond approximately 1,400–1,500 vehicles per hour, with a passing lane closure initiated at approximately 1,800 vehicles per hour. The concept delivered low (4%) improvements, although this required 100% compliance with the posted (lower) speed restriction. At high volumes, driver behaviour changes and utilisation of the passing lane reduces as motorists seek to avoid the delays and frustration caused by the 2-to-1 merge. One of the important findings of the report was that catastrophic merge failure was a limited phenomenon because the traffic flow entering the overtaking facility was restricted to the traffic flow throughput of the upstream single lane section.

The project's assessment of the costs and benefits (operational and safety) of passing lane and 2+1 roadways produced a series of charts which will allow transport practitioners to identify an indicative benefit–cost ratio for a given scheme based on the combination of passing lane length and spacings. The results section shows 'contour plots' which illustrate these relationships for the 15,000 AADT flow range. The full complement of contour plots is provided in appendix A: Option evaluation process.

Abstract

Reducing delay and achieving higher traffic flow rates and a reduction in the frequency and severity of crashes is a key component of New Zealand's long-term success in managing its transport network. The research focused around developing, implementing and measuring safe and robust design principles and techniques to understand economic efficiency and operation on New Zealand's highways with a focus on passing lanes and 2+1 passing facilities.

The research built on existing local and international knowledge, captured key observed data in order to develop an understanding of on-road behaviour and the operational characteristics, identify the core components of economic costs and savings, establish principles for measuring the optimal economic return from the length and frequency, and evaluate the potential benefits of ITS design treatments of passing facilities.

1 Introduction

1.1 Background

The rural New Zealand strategic road network predominantly consists of two-lane two-way roads. This component of the state highway network has an important strategic function – linking urban areas, tourist destinations, playing a key role in the movement of freight and carrying 30% of New Zealand's total annual road travel (vehicle km travelled). Currently a large proportion of this network (around 73% in terms of length) carries lower traffic volumes. In the future a much bigger proportion of the network will carry moderate to higher volumes, 5,000 to 25,000 AADT. It is also anticipated that the proportion of slower vehicles, eg heavy commercial vehicles (HCVs) carrying freight, will increase as the economy grows. The combination of geographic constraints, increasing flow and increasing proportion of slower commercial vehicles places greater emphasis on the suitability and performance of passing facility treatments. At these moderate and higher volumes, passing lanes are generally not treated as isolated facilities, they are considered as a route/corridor treatment with a number of passing lanes placed in series.

Depending on the traffic volume, improvements to traffic operations can include isolated passing lanes at the lower end of the spectrum to access control, four-laning and grade separation at the upper end of the scale. There is a large cost gap between the provision of isolated passing lanes and four-laning, particularly in New Zealand where the topography, poor soil subgrades and 20m road reserve do not easily support cost-effective four-laning.

Economic evaluation is one method used to investigate the merits of a transport intervention. To provide the best value for money for passing facilities as a route/corridor treatment, the interaction between adjacent facilities needs to be considered robust and the resulting efficiency savings balanced against the cost of implementing schemes.

1.2 2+1 roadways

The issue of costs associated with isolated passing facilities through to four-laning has been considered internationally. A number of countries have adopted a three-lane configuration referred to as a '2+1 roadway' at moderate to higher volumes to bridge the gap between isolated passing facilities and four-laning. A 2+1 roadway has alternating passing lanes by direction, often utilising a continuous three-lane cross section for some route segments.

2+1 roadways also provide:

- additional road link capacity (ie between intersections)
- improved journey time reliability
- overtaking opportunity insofar as the provision of overtaking is a fundamental aspect of modern single carriageway design, the requirement for which is accentuated by differential national speed limits associated with different vehicle classes
- 'civilised' driver behaviour on stretches of two-lane single carriageways in order to reduce driver frustration and improve road safety

- an interim measure pending the social, economic and environmental case for (and affordability of) an upgrade to the four-laning standard. In moderate traffic volume situations, 2+1 roadways could be a permanent/long-term intervention.

2+1 roadways have been used in France since the 1960s and have more recently been implemented in other (mostly) European countries.

Despite its lack of New Zealand implementation, the 2+1 roadway is recognised within the New Zealand passing lane strategy (NZ Transport Agency 2007) as an intermediate step to four-laning. It is worth noting that the provision of 2+1 roadway configurations in European countries has not always been as a deliberate means of bridging the gap between two-lane and-four lane carriageways. In Sweden and Germany in particular, the initial 2+1 road configurations were implemented as a means of low-cost improvements to existing wide (13m) two-lane carriageways that had been somewhat unsuccessful, with a poor crash history and prone to unanticipated and undesirable driver behaviour (Potts and Harwood 2003). These roads were converted to 2+1 configurations by re-marking the existing carriageway and, in the case of Sweden, installing a wire rope barrier (WRB or ‘cable barrier’) between opposing traffic lanes.

The typical overseas experience has been that 2+1 roadways improve the level of service for two-lane roads. Implementation of 2+1 roads generally leads to a significant improvement in road safety, resulting in fewer high severity crashes in particular, although internationally the extent of safety of 2+1 roads is more variable where WRBs are not installed.

1.3 New Zealand Passing Lane Strategy

The New Zealand Passing Lane Strategy forms appendix 3e of the *Planning policy manual* (NZ Transport Agency 2007). The key figure is reproduced in figure 1.1. It should be noted that this manual and the New Zealand Passing Lane Strategy is effectively out of date and will likely be superseded by other NZ Transport Agency manuals, strategies or procedures. Thus the quoted strategy should not be relied upon as a specific reference guideline or standard. In the first instance, it is expected that the processes outlined in this research report will in due course be incorporated into the next update of the NZ Transport Agency’s *Economic evaluation manual* (EEM).

Figure 1.1 New Zealand Passing Lane Policy framework (reproduced)

Table App3E/4 - Long-term framework for passing and overtaking treatments

<table>
<tr><th rowspan="2">Projected AADT (vpd)</th><th colspan="3">Road Gradient</th></tr>
<tr><th>Flat</th><th>Rolling</th><th>Mountainous</th></tr>
<tr><td>0-2,000</td><td colspan="3">Overtaking (OT) (OT sight distance improvements, OT enhancements, possible isolated shoulder widening/crawler shoulder/SVBs[1]/short PLs.</td></tr>
<tr><td>2,000-4,000</td><td>Overtaking (As above).</td><td colspan="2">Mainly OT, as above but possibly some SVB[1] or short PLs @ 10 km.</td></tr>
<tr><td>4,000-5,000 (General transition to PLs)</td><td>Mainly OT, as above but possibly some SVBs[1] or short PLs @ 10 km.</td><td>PLs @ 10km 1.2 km + tapers & OT enhancements.</td><td rowspan="2">PLs @ 5 km 1 km+tapers & possible OT enhancements.</td></tr>
<tr><td>5,000-7,000</td><td colspan="2">PLs @ 5 or 10 km[2] 1.2 km + tapers & OT enhancements.</td></tr>
<tr><td>7,000-10000</td><td colspan="2">PLs @ 5 or 10[2] km 1.5 km + tapers & OT enhancements.</td><td>PLs @ 5 km 1.2 km+tapers & possible OT enhancements.</td></tr>
<tr><td>10,000-12,000 (General transition to 2+1 lanes)[3]</td><td>PLs @ 5 km 1.5 km + tapers & possible OT enhancements</td><td>2+1 lanes (subject to four-lane comparison)</td><td rowspan="3">PLs @ 5 km 1.2-1.5 km + tapers.</td></tr>
<tr><td>12,000-20,000</td><td colspan="2" rowspan="2">2+1 lanes (subject to four-lane comparison).</td></tr>
<tr><td>20,000-25,000 (General transition to 4 lanes)</td></tr>
</table>

Key – strategy type	**Overtaking**	**Mainly overtaking**	**Passing and overtaking**	**Passing**

Notes: 1. Where appropriate, a SVB is able to be easily altered to a short PL or PL at a later date.
2. Along the same road section, a mixed layout with 5 km spacings in higher demand locations and 10 spacings in lower demand locations.
3. For flat or rolling road gradient, the combination of passing lane length and spacing may not be sufficient to dissipate vehicle queues and a more frequent provision of passing opportunities would be required. Therefore, passing treatments, such as 2+1 lanes (subject to comparison with four-lanes), are likely to be required for state highways with a flat or rolling gradient and projected 10,000-25,000 vpd.
4. 10,000-12,000 vpd represents a general upper limit for passing lanes in series with flat or rolling gradient. Above this threshold, treatments such as 2+1 lanes (subject to comparison with four-lanes), are likely to be required. Some locations may have a higher upper limit of about 14,000 vpd depending on other factors, such as proportion of directional flow and traffic composition.

As can be seen, the above strategy suggests roads with a volume of up to 10,000 vehicles per day (vpd) rely on overtaking opportunities and those created by isolated passing bays. Beyond 25,000vpd there is a general transition to four lanes. This broadly leaves a band between 10,000vpd and 25,000vpd in which more exploration is required into route/corridor treatment of passing strategies.

1.4 Research objectives

The core aspects of this research relate to the merits of 2+1 configurations. The NZ Transport Agency (Transport Agency) has carried out specific research into the relative benefits of different cross sections, and this will be used to inform subsequent decisions around form and design of 2+1 layouts. Commentary and information provided in this research report in relation to 2+1 design, configuration, cross-section layout etc are therefore not considered definitive but will inform the final Transport Agency design guidelines.

The research for this project, which commenced in November 2011, can be applied to New Zealand's rural state highways and has established economic optimisation principles and influential factors which affect the characteristics and operation of passing lane treatments in series at moderate to higher volumes across the various terrains within New Zealand. The research objectives were as follows:

- Identify the likely operational benefits of high-volume passing lane treatments in series and 2+1 layouts under a range of road/traffic parameters using robust survey data and microsimulation traffic modelling.
- Consider the design and safety performance of New Zealand passing lanes and international 2+1 roadways to develop a New Zealand 2+1 roadway design and determine the likely safety characteristics of 2+1 roadways in the New Zealand context.
- Understand driver merging behaviours and assess a range of ITS-assisted merging techniques.
- Develop a draft EEM procedure which considers the operational benefits, safety benefits and cost implications to establish optimal passing lane length and spacing combinations across a range of traffic and terrain variables.

1.5 Report structure

Chapter 2 of this report summarises the operation and construction of 2+1 roadways, outlining potential issues and characteristics experienced internationally.

Chapter 3 describes the safety characteristics of passing lanes and 2+1 roadways.

Chapter 4 discusses 2+1 roadway design options and proposes a New Zealand 2+1 standard.

Chapter 5 describes the data collection process which is reliant on robust microsimulation traffic modelling.

Chapter 6 details the traffic modelling methodology.

Chapter 7 discusses the development, testing and results from the range of ITS-assisted merge concepts.

Chapter 8 considers the economic evaluation component through assessment and comparison of the costs and benefits.

Chapter 9 describes the results of the extensive traffic modelling activities.

Chapter 10 details the conclusions and recommendations.

The processes outlined in appendix A have been designed for practitioners to develop an economic evaluation with a benefit-cost ratio (BCR) for passing lane treatments in series (including 2+1 layouts) on flat/rolling road gradients, and high-volume passing lanes in series on mountainous road gradients. The processes are applied separately in each direction of travel to two-lane rural, 100km/h and state highway environments for two-way AADTs between 5,000 and 25,000vpd.

2 2+1 roadway operations

In the absence of many 2+1 installations in New Zealand, a review of international experience has been undertaken to understand and inform the useful characteristics of their operation. The project's literature review identified 2+1 configurations in the following countries:

- Austria
- Denmark
- Finland
- France
- Germany
- Ireland
- Italy
- The Netherlands
- South Korea
- Spain
- Sweden
- United Kingdom (including Scotland).

The information available for the various international examples varied considerably. For this reason, only those with a sufficient level of detail/context have been further considered in this report. Table 2.1 provides a summary of 2+1 operational aspects from overseas (from various sources) which broadly covers the range for further investigation in New Zealand.

Table 2.1 Comparison of international 2+1 operational characteristics

Country	Speed limits	AADT ranges 2-way (vpd)	Estimated capacity 1-way (vph)
UK	–	10,000 – 25,000	
Ireland	80–100km/h	11,000 – 18,000	
Germany	100km/h	7,000 – 25,000	
Finland (theoretical)	–	<14,000	
Sweden	90–100km/h	4,000 – 20,000	1,600 –1,700vph
Denmark	80–90km/h	7,000 – 15,000	
Austria (proposed)	≤100km/h	7,000 – 18,000	1,500vph
Overall	**80–100km/h**	**4,000 – 25,000**	**1,500 – 1,700vph**

2.1 Speed limits

In Sweden, the speed limit used for 2+1 roads has historically depended on the type of access control, with 'semi-motorways' (ie grade separated intersections) having 110km/h limits and 'normal' roads (ie at-grade intersections) having 90km/h limits (Bergh and Petersson 2010). More recently they have all been posted at 100km/h irrespective of the intersection configurations. The justification for these changes is not adequately recorded in the literature.

Bergh et al (2005) noted that 2+1 roads with 110km/h limits typically had a 15% to 20% higher rate of crashes than their 90km/h counterparts. Sandle et al (2005) also suggested that the layout could encourage excessive speed, so any existing speed limits might need to be reviewed.

2.2 Traffic volumes and capacity

Although 2+1 roadways have generally been pitched in the region between normal two-lane roads and four-lane expressways, it is not completely clear which are the most appropriate traffic volumes for 2+1 options. As indicated by table 2.1, many sites with AADT below 10,000vpd or above 20,000vpd have been considered appropriate for 2+1 installations. This is somewhat dictated by local differences in crash rates, construction costs, assessed benefits and political/technical preferences. Construction costs are particularly notable – in many international locations existing two-lane roads included a more substantial road reserve and high-standard hard-shoulder. This makes retrofitting of 2+1 layouts to the existing carriageway relatively cheap, and hence economically viable at lower traffic volumes.

There have been some concerns expressed that 2+1 roadways could result in reductions of speed and capacity due to the introduction of no-overtaking restrictions on the single-lane sections (whether by means of a physical barrier or just no-overtaking markings). However generally this has not been found to be the case, with the regular overtaking provision in the passing lane sections more than compensating. Bergh et al (2005) noted for example that new 90km/h 2+1 roadways had seen average journey speeds increase by about 2km/h. Good level of service was still observed at higher traffic volumes and during hourly volume peaks.

2.3 Effect on platooning and speed

Potts and Harwood (2003) modelled a series of passing lane configurations (including a 2+1 layout) with different traffic volumes and directional splits. Compared with a conventional two-lane road, a 2+1 layout typically provided a level of service rating (A to F) at least one or two levels higher. This typically translated to an increased mean vehicle speed of ~2km/h and a reduction in the percent of time spent following of 15% to 28% (more at higher volumes). Similarly, Sandle et al (2005) found that a retrofitted 2+1 road in Scotland resulted in no significant change in journey speed compared with the previous two-lane road. Frost and Morrall (1995) also found that 2+1 roads in Germany typically experienced ~15% reductions in the percent of vehicles following compared with a standard two-lane road.

Durth (1995) noted that German 2+1 roadways hardly showed any difference in speed behaviour compared with their previous 'wide' two-lane counterparts. He did suggest, however, that the placement of a solid median barrier on an existing four-lane road resulted in 'slightly lower' speeds but without detrimental effects on traffic operation. One could speculate whether a less intrusive WRB would have a similar or negligible effect on travel speeds. Experience of the State Highway (SH) 1 Longswamp to Rangiriri 2+1 roadway suggests that proximity of the barrier influences travel speed in a single lane.

Enberg and Pursula (1997) modelled various passing lane combinations and found that the overtaking rates for 2+1 layouts were consistently higher than for single passing lanes, especially at high flow rates. Interestingly, when they investigated the effect of passing lane length vs traffic flow they found there was virtually no difference in overtaking rates for passing lane lengths between 1km and 2.5km; however, there was a slight increasing relationship with traffic flow. Enberg and Pursula also found increases in speeds on the three-lane sections, with a 1km/h to 3km/h increase in free speeds rising to 3km/h to 5km/h at higher volumes.

Irzik (2010) investigated the effect of traffic volume on the length required to disperse a platoon. Typically at least ~1,000m was required for volumes of 400veh/hr (one direction), rising to ~1,700m as traffic flows doubled. Irzik also noted that greater rates of %HCVs in the traffic stream lessened the required passing lane length, by approximately 50m for every 5% of HCVs. A conventional diverge (where the overtaking lane is added to the right) also required ~75m less than a lane gain obtained from an entry ramp, where slower vehicles might have to move over to the new slower lane.

2.4 Miscellaneous considerations

As well as the main design, operational and safety considerations of 2+1 roadways, the literature has also identified other issues that need to be considered in the planning and design of these roadways.

2.4.1 Maintenance

2+1 layouts may provide some different issues in terms of maintenance, particularly for those with median barriers. Any operations that block the single-lane section could have significant consequences, so maintenance works on the single lane either need to continue to provide sufficient passing width or be undertaken during low-traffic periods. Works that could be undertaken on either side (eg repair/ replacement of barrier posts) are usually carried out by closing the overtaking lane.

Sandle et al (2005) also noted that maintenance agencies had reported difficulties setting out temporary traffic management for routine maintenance in the vicinity of the transition zones. For some activities, closures of one direction at night-time had been necessary to achieve the required work.

2.4.2 Slow vehicle access

2+1 roadways provide a high-speed experience, which is not particularly compatible with lower-speed vehicles, such as agricultural equipment and bicycles. This is particularly an issue if the cross-section design provides little in the way of a shoulder and possibly a median barrier restricting the ability to overtake in the single-lane sections.

In most cases overseas, it appears that efforts are generally made to provide alternative access for such vehicles away from the 2+1 roadway, eg by means of parallel local roads or off-road pathways. Durth (1995) categorised 2+1 roadways as being ‘for motorised traffic only’.

Brilon and Weiser (1995) cited new German guidelines providing warrants for when pedestrian and bicycle traffic should be provided with a separate path. The figures are reproduced in table 2.2.

Table 2.2 Warrants for separate pedestrian/bicycle paths (Brilon and Weiser 1995)

Motorised traffic (veh/day)	Separate path for pedestrians: volume of pedestrians (peds/peak hour)	Separate path for cyclists: volume of bikes (bikes/peak hour)	Combined path for pedestrians/bikes: volume of pedestrians + bikes (peds-bikes/peak hour)
<2,500	60	90	75
2,500 – 5,000	20	30	25
5,000 – 10,000	10	15	15
>10,000	5	10	10

If only daily traffic volumes for pedestrians and bikes are available, the peak hour is considered to be 20% of the daily volumes.

2.4.3 Emergency vehicle access

2+1 roadways with median barriers cause some concerns about the ability to readily access the other side of the road should the need arise by emergency or maintenance vehicles. Two strategies addressing this were noted by Bergh and Carlsson (2000):

- In Sweden 'quick locks' were placed in the WRB at each transition zone to enable opening of the barrier. This would allow for rapid reconfiguration of a 2+1 road, eg to close a single-lane section.
- Permanent emergency openings were established every 3km to 5km to allow rescue vehicles to turn.

One of the key objectives of the research was to determine the safety and operational benefits for 2+1 layouts and high-volume passing lanes in order to develop the processes set out in appendix A. The safety characteristics of passing facilities are directly related to the infrastructure design. The similarities between passing lanes and 2+1 layouts (consecutive, alternating passing lanes) make consideration of passing lane design a key input to safety.

2.5 Cross-section design

The standard layout of passing lanes is given in the *Manual of traffic signs and markings* (MoTSaM) (NZ Transport Agency 2010a). This provides a consistent design that is applied throughout the country.

The design anticipates a 'tack-on' style passing lane, whereby the widening is provided on the outside of the carriageway with the centreline being unaffected. All traffic lanes are required to be a minimum of 3.5m, with shoulder widths being retained at the same width within the road section, meaning that the widening required to implement a passing lane in one direction is that of the lane, being 3.5m. With state highways in New Zealand generally having shoulders of between 0.5m and 1.5m, this indicates a total carriageway width requirement of 11.5m to 13.5m where there is a passing lane in one direction only. Passing lane design and (uni-directional) 2+1 roadway designs have common features.

Table 2.3 provides a summary of 2+1 carriageway cross-section designs from overseas (from various sources).

Table 2.3 Comparison of international 2+1 roadway cross sections

Country	WRB	Sealed shoulder per direction (m)	Dual lane widths (m)	Single lane width (m)	Median width (m)	Total sealed width (m)
UK (design standard)	No	1.0	3.5	3.5	1.0	13.5
UK (constructed)	No	1.0	3.5	3.5	0.75	13.25
Ireland	Yes	0.5–1.0	3.25–3.5	3.5	1.25–2.0	12.25–14.5
Germany	No	0.25	3.25–3.5	3.5–4.25	0.5	11–12.25
Finland	No	1.25	3.25–3.5	3.75	0.0	12.75–13.25
Finland	Yes	0.9–1.25	3.25–3.5	3.75	1.7	13.75–14.95
Sweden	Yes	0.5–1.0	3.25–3.5	3.5–3.75	1.25–1.75	12.25–14.5
Austria (proposed)	No	0.5	3.25–3.75	3.5–3.75	0.75–1.0	11.75–13.25
Denmark	No	1.0	3.25	3.5	0.0	12.0
France	No	0.25	3.5	3.5	0.0–0.5	11–11.5
South Korea (proposed)	No	1.5	3.25	3.5	0.5–1.5	13.5–14.5
Overall	**Varies**	**0.25–1.5**	**3.25–3.75**	**3.5–4.25**	**0–2.0**	**11–14.95**

As can be seen, there is some variation in the European carriageway width provisions. The minimum width identified is 11m and the maximum is nearly 15m. It is unlikely that an 11m wide configuration would be provided in New Zealand because it is based on narrow lanes or shoulders that would not meet New Zealand lane and shoulder standards. At the wider end of the scale, carriageways of 14+m are provided in Sweden, Finland and Ireland, with the provision of a WRB being the key difference in the overall width when compared with narrower configurations.

In Germany, in particular, many 2+1 roads were implemented as a cost-effective means of removing existing two lane roads with wide shoulders or wide traffic lanes, within an overall carriageway width of 1m to 12m. Such wide rural two-lane carriageways do not generally exist in New Zealand. In these situations the carriageway width was not determined through the design, rather the design was fitted to the existing carriageway width. Hence, the narrower widths should not necessarily be considered as a 'design standard'.

The New Zealand situation is probably more akin to the development of the 'wide single 2+1' (WS2+1) design in the UK, which was trialled at sites in Scotland (Sandle et al 2005). The existing two-lane routes were typically 10m wide, thus additional widening was needed to create the WS2+1 profile; this was generally developed on one side of the existing centreline so that the crown of the road was still between the opposing traffic.

It is notable that a number of European countries have not been concerned about the placement of the existing road crown when a carriageway is re-marked as a 2+1 roadway. In some cases this is because standard practice is to provide a constant cross-fall across the carriageway without a crown (eg Germany). However, Potts and Harwood (2003) note that in Sweden no attempt was made to adjust the lane positions relative to the existing crown; hence typically the crown falls within the centre lane. No safety difficulties have been noted with this approach.

Consideration of future four-laning may also play a part in determining cross-section dimensions to use. Two potential strategies appear to be:

- Construct the passing lane component in a location suitable for the ultimate construction of the four-lane carriageway, and widen the single-lane section later. This is particularly relevant where a central WRB has already been installed as part of the layout.
- Construct a completely separate second carriageway later, and reconfigure the 2+1 carriageway to be a wide two-lane carriageway in the opposing direction. Schermers et al (2010) advocated this approach for the development of 2+1 roads in the Netherlands.

A number of design standards suggested that regular breakdown shoulders or 'lay-bys' should also be incorporated (if median barriers are used), eg Hofbauer (2010) recommended at least one per single-lane section, with Durth (1995) recommending that they be located in the middle of the single-lane section.

2.5.1 Cross-section summary

A summary of key points from the review of international of 2+1 roadways is as follows:

- Same or wider lane/shoulder widths are used on the single-lane side, often to provide sufficient space for a broken-down/crashed vehicle.
- Many 2+1 configurations have been developed from existing 'wide' (11m+) two-lane roadways, hence the use of narrow lane/shoulder widths to fit everything in. This is not generally the same situation in New Zealand (few two-lane roads are >10m wide).
- Provision of a median barrier, mainly WRB, generally requires additional median space.
- Location of road crown is generally not an issue when re-marking existing roads (some countries do not have a crown); no safety issues were noted.
- Different possible future strategies could be to widen the existing 2+1 road for four-laning later or to construct a separate second carriageway.
- Regular breakdown shoulders or lay-bys could also be incorporated in the single-lane section where median barriers are provided.

2.6 Passing lane length

The specification of passing lane length on a 2+1 road is an interesting constrained problem, because it affects both the passing lane and single lane sections. As pointed out by Enberg and Pursula (1997), a longer passing lane section allows for more reduction in platooning of traffic; however, the rate of reduction will typically drop off with distance. Meanwhile a longer single-lane section, without opportunity for overtaking, will increase the proportion of vehicles platooned waiting to overtake in the opposing direction. Hence, there will be an equilibrium point at which the benefits provided to the passing lane traffic no longer exceed the disbenefits suffered by the single-lane traffic.

Irzik (2010) noted that safety is often a reason for specifying minimum lengths for passing lanes. He cited another study where passing lane sections shorter than 1,000m had a disproportionately high number of crashes compared with longer passing lane sections.

Durth (1995) found that, for volumes up to 1,000veh/hr in both directions and an average HCV% up to 15%, passing lane section lengths between 1,000m and 1,400m were best. Higher proportions of HCVs favoured shorter lengths, due to the greater speed differentials.

Potts and Harwood (2003) recommend that passing lane lengths for 2+1 roads should be consistent with the optimal lengths determined for isolated passing lanes; however, no modelling was done to confirm this. Szagala (2005) modelled a series of 2+1 roads with varying passing lane lengths between 600m and 2,000m and a range of traffic volumes from 250 to 1,500veh/hr each way. The report concluded that, for passing lanes 950m to 2,000m in length, there was no significant difference in percentage of time spent following for the whole range of traffic volumes analysed. Thus in practice passing lane lengths between 1,000m and 2,000m can be used for 2+1 roads, regardless of traffic volume.

Table 2.4 provides a summary of 2+1 longitudinal designs from overseas (from various sources). The table also identifies critical and non-critical transition lengths which are discussed further in section 2.7.

Table 2.4 Comparison of international 2+1 section lengths (dimensions in metres)

Country	Typical section lengths	Absolute minimum length	Absolute maximum length	Non-critical (diverge) length*	Critical (merge) length*
UK (design standard)	0.8–1.5km	0.6km	2.0km	50m	300m
Ireland	1.0–2.0km	0.8km	3.0km	50m	300m
Germany		1.0 km	1.4 km	≥30m	180m
Finland	1.5km			50m	500m
Sweden		1.0 km	2.5 km	100m	300m
Austria (proposed)	1.2–1.8km	1.0 km	2.0 km	90m	300–400m
Denmark	1.55km				300m
S. Korea (proposed)	1.0–1.5km			90m	280m
Overall	**0.8–2.0km**	**0.6–1.0km**	**1.4–3.0km**	**30–100m**	**180–500m**

*Note: these lengths include a taper in each direction and buffer zone in some instances. The passing lane lengths used throughout this report do **not** include any tapers.

2.6.1 Passing lane length summary

A summary of key points associated with the passing lengths of 2+1 roadways is as follows:

- While longer passing lanes allow for more reduction in platooning of traffic; the rate of reduction will typically drop off. Longer single-lane sections, without an opportunity for overtaking, will increase the proportion of vehicles platooned waiting to overtake. Hence, there will be an equilibrium point at which the benefits to the passing lane traffic no longer exceed the disbenefits suffered by the single-lane traffic.
- Unlike isolated passing lanes, where increased traffic volumes usually result in longer optimal passing lane lengths, there may be relatively little difference in time spent following for 1,000m to 2,000m passing lanes used in 2+1 configurations.
- Some countries employ a minimum passing lane length, usually for safety reasons (too many late overtakings); less than the 1,000m typically not recommended.

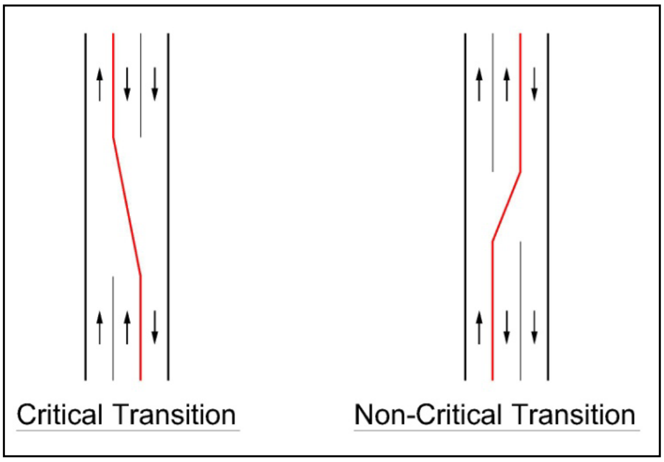
- Slightly shorter passing lane lengths may be used where HCVs% are higher, as higher passing rates can be achieved when passing slower vehicles.

2.7 Merge and diverge design

Existing New Zealand passing lane standards, based on a 100km/h speed limit, require a diverge length of 100m. Also based on a 100km/h design speed, the length of the merge area is some 335m, consisting of a 160m edge line taper followed by the edge of seal tapering back to the standard width over 175m (1:50). A continuity line is used to demarcate the left-hand lanes through the diverge section, guiding drivers into the left lane unless passing. A similar line is not used at the merge, although chevron markings are provided from the start of the edge line merge taper to the end of the edge of seal taper.

Reference to critical and non-critical transitions between passing lane sections is made within the 2+1 roadway literature. A ‘critical transition’ is the merge section of the roadway between passing lanes. The term ‘critical’ is used because merging vehicles in the central lane are travelling toward each other. A ‘non-critical transition’ is the diverge section of the roadway, where vehicles in the central lane at the ends of the transition are travelling away from each other. This is shown in figure 2.1.

Figure 2.1 2+1 passing lane transitions



Due to vehicles in the central lane travelling towards each other in a critical transition, the length is significantly longer than is provided within a non-critical transition. European examples involve non-critical transition lengths ranging from 30m to 100m, and critical transition lengths of 180m to 500m. Adopting the current New Zealand design for standard passing lanes, it would be expected that transition lengths for 2+1 roadways within New Zealand would be toward the upper limit of those presently used in European countries. Austroads recommends that the ends of passing lane merges are separated by three seconds of travel time. At 100km/h, in combination with the standard 160m merge tapers, this would suggest a critical transition length of 400m.

Durth (1995) noted a number of key criteria for determining the location of transitions:

- Locate the transition where the driver can concentrate on their own lane or oncoming traffic.
- Do not locate where slippery conditions might prevail, eg on bridges in winter.
- On tight bends the passing lane section should be on the outside of the curve; if this is not possible it may be better to temporarily abandon the three-lane layout.

- A passing lane should be provided for uphill traffic.
- For safety reasons, single lane sections should be used for cross-town links where state highways pass directly through rural villages and towns.

New Zealand's current passing lane merge and diverge layouts are based on driver behaviour research undertaken using a driving simulator (Luther et al 2004). One operational factor that needs to be considered when reviewing European 2+1 designs is that the merge and diverge markings may differ. For example, Frost and Morrall (1995) noted that with German passing lanes the high-speed (overtaking) lane is terminated with vehicles merging into the slower lane; this contrasts with Canadian passing lanes where both lanes are expected to merge equally together.

Herrstedt (2001) in a review of Danish trial 2+1 roads (where the overtaking lane is also terminated at the merge), noted that only about 2% to 6% of vehicles overtake on the last 100m of the passing lane section. However, the field measurements indicated that increasing road traffic was leading to an increased proportion of vehicles overtaking on the last 100m of the passing lane section.

For several reasons, 2+1 roadways in New Zealand are unlikely be constructed as a continuous width carriageway, as they are in Europe. Due to 2+1 roadways in New Zealand involving the widening of an existing carriageway rather than retrofitting an existing wide carriageway, there is no reason to retain the three-lane roadway width within the transitions or for the passing lanes to be immediately back to back other than in high-volume situations where it is economically viable for back-to-back passing lane length/spacings. In fact it is likely that passing lane transitions would be selected around constraints including: intersections, clusters of property accesses, horizontal curves, bridges/other width constraining features. In these sections, the roadway would reduce to a standard two-lane carriageway between passing lane sections and the existing diverge and merge standards would apply.

2.8 Markings and signage

Figure 2.6 'Markings for passing lanes' from MoTSaM (NZ Transport Agency 2010a) provides the standard geometric and marking arrangements for passing lanes within New Zealand.

One difference between standard New Zealand passing lanes and European 2+1 roadways is the treatment of the centre line. With traditional passing lanes, the road centreline is effectively unchanged from the general two-lane carriageway, with widening facilitating the passing lane being provided on the outside of the carriageway – a 'tack-on' passing lane. International 2+1 designs, however, are based on a carriageway with a consistent width that does change over the length of the road. In this situation it is the centreline that shifts to allow the two-lane section to alternate between directions of travel. The effect on markings is that in traditional passing lanes, the left-hand lane merges into the right-hand lane, whereas in 2+1 roads the central lane merges into the left-hand lane. These differences are unlikely to result in significant variation in construction costs.

In terms of separation between opposing lanes, the current practice in New Zealand is to provide a 'no overtaking' line between opposing traffic lanes in the direction of travel of the passing lane. This results in a double yellow line between opposing traffic lanes where there are passing lanes in both directions. In a three-lane situation, the New Zealand practice is less onerous than that recommended by Austroads (2009), which states the double two-way barrier lines (equivalent to New Zealand's double yellow line) are required 'to mark the dividing line on three-lane overtaking lane sections on road. If visibility is sufficient, a double one-way barrier line may be used in some circumstances'. This suggests that the Austroads approach is to default to a double yellow line (white in Australia), whereas the New Zealand approach is to default to a single yellow line in the direction of the passing lane in a three-lane situation.

The presence of no-overtaking markings may not be sufficient to deter overtaking in the single lane direction. Sandle et al (2005) observed at 2+1 roads in the UK, that some drivers still made illegal overtaking manoeuvres (especially to overtake slower agricultural vehicles), particularly where forward sight distance was very good.

2.9 Grade and terrain

In New Zealand, the majority of two-lane rural highways, where 2+1 designs may be appropriate, are near the major urban centres where traffic volumes are higher. The terrain of these roads is generally rolling to flat. The more mountainous stretches of the strategic network generally have lower volumes that are unlikely to justify a 2+1 treatment, and where general overtaking opportunities and isolated passing lanes will continue to be appropriate provisions. Therefore, it is unlikely that terrain would be the critical factor in determining if a 2+1 roadway should be installed.

It is noted that MoTSaM differentiates between slow vehicle bays and passing lanes. While there are certain circumstances when slow vehicle bays may be appropriate, passing lanes are the preferred treatment as drivers are more familiar with them (NZ Transport Agency 2010a). Hence, even in mountainous terrain, passing lanes are a preferred treatment within New Zealand.

When considering the relative location of the passing lane and single lane sections, a number of design guides elsewhere recommend placement of the passing lane section on the ascending gradients, eg Hofbauer (2010).

Interestingly, Sandle et al (2005) suggested that a 2+1 layout had a significant advantage over conventional two-lane roads in reducing ‘environmental impact’ due to the reduced requirement to provide overtaking sight distance for passing in the opposing lane. Given that a 2+1 alignment would presumably still require adequate stopping sight distances (and at possibly higher design speeds) it is not clear whether this provides a real advantage, although overtaking sight distance would be greater than stopping sight distance.

2.10 Intersection design

2+1 roads provide a transition between traditional two-lane (single-carriageway) roads and four-lane (dual-carriageway) expressways. As such, they straddle the types of intersections provided along these types of roads, typically at-grade priority or roundabout intersections on the former and grade-separated ‘interchanges’ on the latter.

Where an at-grade intersection is being provided or retained, design guidance elsewhere (eg Highways Agency 2008) was almost unanimous in recommending that such intersections not be provided *within* the 2+1 sections, but rather *between* such sections, either in two-lane (1+1) sections or in transition zones. In the latter case, Hofbauer (2010) points out that the most appropriate location for intersections is at a non-critical transition. Some guidance (eg Durth 1995) went further in recommending that *all* intersections on 2+1 sections should be grade separated for safety.

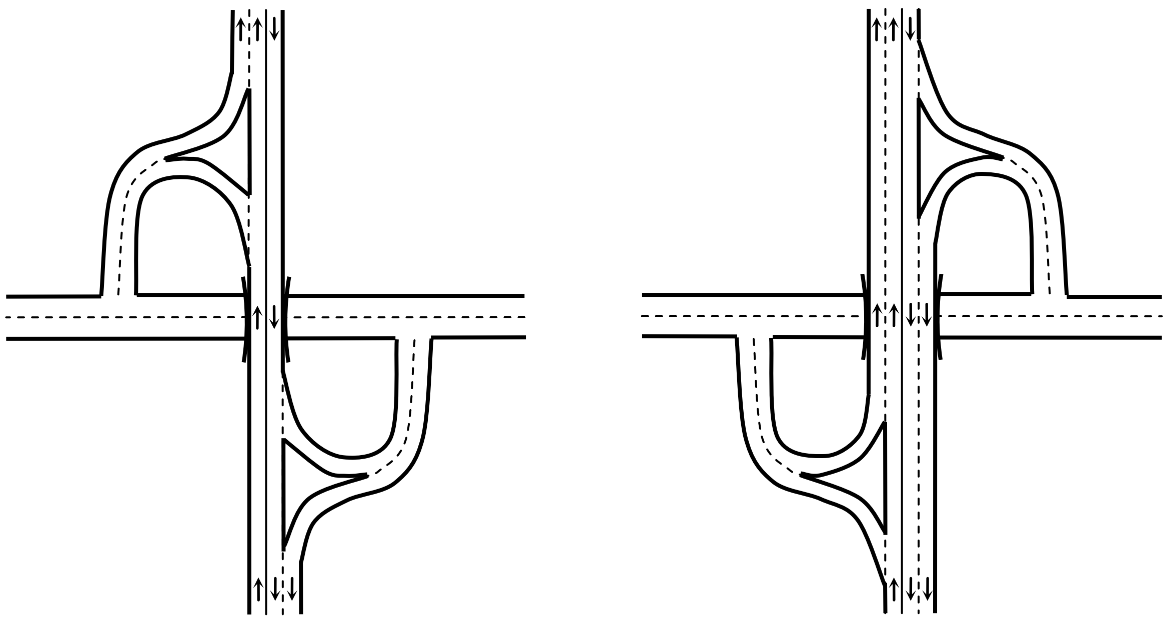
Weber (2005) noted that the relative severity of crashes near intersections was higher than elsewhere along 2+1 routes in Germany, with almost half of the crashes being of a 'turning/crossing' nature. At-grade intersections had crash numbers more than double their grade-separated counterparts. Three intersections that were signal controlled were among the worst, highlighting the difficulties in using this form of control on high-speed roads.

As traffic volumes increase, grade-separated interchanges are more warranted due to safety concerns. Koorey (2009) noted that, while only 8% of crashes on rural two-lane roads occurred at intersections, they made up 23% of crashes on three-lane and four-lane undivided rural roads. Common designs in Europe allow for motorway-style entry and exit ramps on the 2+1 road connecting to simple priority or signalised intersections on the minor road. The layout of a grade-separated interchange can also be deliberately designed to take into account the cross-section of an overbridge for the main road. If only a two-lane overbridge is provided (to minimise costs) the exit ramps can be configured to start new 2+1 sections either side of the bridge structure. Conversely, if the design wants to incorporate a four-lane bridge structure now (to future proof for later widening) the exit ramps can be configured to incorporate the extra lanes across the bridge.

This discussion is somewhat of an over-simplification of the challenges posed by addressing how 2+1 roadways might interface with existing or new/improved accesses/intersections. Another key component to this discussion will be determined by the desired location of the passing lane itself (for example based on AADT, terrain, topographical constraints and %HCV)

Figure 2.2 illustrates these two possible layouts, the main road is shown running north-south and the side road east-west.

Figure 2.2 2+1 interchange layouts (schematic, not to scale)



Sandle et al (2005) noted that in some cases, if there are close intersection spacings and/or the presence of accesses, then such sections may not be suitable for conversion to 2+1 roadways.

3 Passing lane safety

3.1 Base line crash statistics for rural two-lane two-way roads

As will be shown in subsequent sections of this report, crash studies associated with passing provisions often base crash reductions as a proportion reduction of existing crashes. To quantify these reductions for economic evaluation requires a baseline crash rate. A summary of typical crash rates (per 10^8veh-km) for two-lane, two-way rural roads to establish this baseline has been made using the EEM and is provided in table 3.1

Table 3.1 Baseline coefficients for rural mid-block crashes in New Zealand (reproduced from NZ Transport Agency 2010b)

AADT (vpd)	Mean seal width (m)	Coefficients b_0 by terrain type		
		Level (0 to 3%)	Rolling (>3 to 6%)	Mountainous (>6%)
<1,000	6.7	16	21	30
1,000 – 4,000	8.2	16	18	26
>4,000	9.5	11	16	22

These base rates are then adjusted to account for differences in traffic lane and shoulder widths. For example, for roads with traffic flows >4,000vpd, a cross-section with 3.5m traffic lanes and 2.0m shoulders would have a crash rate of only 66% of the base rate, whereas a road with 3.25m traffic lanes and 0.5m shoulders would have a crash rate 167% of the base rate.

3.2 Passing lanes crash reduction

Table A6.18(d) of the EEM provides typical crash reductions for mid-block treatments in high-speed areas. The reductions for passing lanes are as follows:

- 30% of overtaking crashes within the passing lane
- 40% to 60% of head-on crashes within the passing lane
- 15% of rear end/obstruction crashes within the passing lane.

Reduce these percentages linearly to zero for crashes following the passing lane up to 5km away.

These reductions are considered to represent a 25% reduction in overall crashes within the passing lane, which is adopted for crash rate analysis within section A7 ‘Passing lanes’ of the EEM. This is consistent with national research findings (NZ Transport Agency 2010b), which showed three-lane highways operate with 27% fewer injury crashes than two-lane highways. This is averaged over all terrain categories. Note that this reduction applies to the section of road, rather than the direction with two lanes, and does not account for downstream benefits.

Koorey et al (1999) has shown that crash reductions are observed to be relatively evenly split between traffic travelling in each direction, with no evidence to suggest that the majority of crash savings occur in the direction of travel served by the passing lane. This identified that the main downstream benefits occur 4km to 8km downstream of the end of the passing lane. It also indicated that there is an increase of crashes immediately (0km to 2km) downstream of a passing lane, which could reflect problems associated with completing merging manoeuvres and slowing down from overtaking speeds. No clear crash reduction trends are identified in terms of AADT.

3.3 2+1 road safety from overseas

A motivation for many of the overseas implementations of 2+1 roads was the fact that alternatives such as 'wide' two-lane roads (where the traffic lanes and/or shoulders were wider than normal to facilitate passing) were not as successful as hoped in terms of safety. Durth (1995) notes for example that extra-wide two-lane roads in Germany had crash rates 40% to 80% higher than their 2+1 replacements, while undivided four-lane roads had rates 75% higher.

Table 3.2 summarises international crash reductions associated with 2+1 roads; where possible these reductions have been identified in comparison to a typical 'normal' two-lane road (ie those without unusually wide lane/shoulder widths).

Table 3.2 Comparison of international 2+1 roadway safety performance

Country	Length of road	AADT	Crash reduction			WRB Installed
			Fatal	Fatal + injury	All (including non-injury)	
Sweden	950km	4–20,000	50%+	40%	increase	**Yes**
Finland (theoretical)	–	-	46%	25%		**Yes**
	48km	<14,000	0%	11%		No
Germany	360km	15–25,000		36%	28%	No
Denmark	24km	7–15,000	67%	0%	(+16%)	No
US (theoretical)	–	–	–	24%		No
Combined average			48%	33%		**Yes**
			33%	21%		No

The average 21% reduction overall outlined above compares closely with the 25% typical reduction in crashes attributed to the construction of a passing lane in New Zealand. Of note is that the 25% passing lane benefit in New Zealand applies to the section of road where the passing lane is constructed. In any 2+1 scheme it is only possible to have three lanes over a proportion of the scheme length. If a 10% to 15% allowance is made for the merge and diverge areas, intersections and tight curves that preclude three lanes, then the actual three-lane sections generating the safety benefits would be nearly identical to the 25% typical New Zealand crash rate.

Potts and Harwood (2003) cited concerns from Finland about the safety of the merge transition zone immediately downstream of the passing lane section. Higher speeds in the passing lane sections (compared with the single-lane sections) and late passing manoeuvres were noted as having contributed to head-on collisions. Coupled with the relative lack of improvement in fatality crash numbers, this had prompted Finnish authorities to recommend median barriers in subsequent 2+1 installations. Sandle et al (2005) also observed various conflict manoeuvres within the critical transition zones of UK 2+1 roads, with late overtaking of slower-moving vehicles continuing throughout the entire hatched merge zone.

3.4 Wire rope barriers and 2+1 passing lanes

WRBs have not traditionally been implemented as part of passing lane installations within New Zealand. However, this is changing as a result of implementation of the 'Safe System' approach to road safety and the government adopting the *Safer journeys* strategy in 2010 (Ministry of Transport 2010). The merits of other forms of median treatments, eg wide centrelines, and median widths are currently being researched separately. Overseas, there is a varied approach to the provision of WRBs on 2+1 roadways, as indicated in table 2.3 (chapter 2). The National Road Authority (2004) described how, in Ireland, where WRBs are used, the median barrier is the prime motivator for the 2+1 installations rather than the passing lanes themselves. In other words, the barriers are desired to prevent head-on crashes and restrict turning movements, but on a two-lane carriageway this would remove passing opportunities. The 2+1 layout provides a mechanism where the barrier can be provided and passing opportunities retained. Bergh et al (2005) also observed that the bulk of the safety benefits from Sweden's 2+1 roads appeared to be due to the WRBs.

In Europe, Sweden uses cable barriers to separate lanes, placing a high importance on the barrier for safety (Larsson et al 2003). Potts and Harwood (2003) noted that Germany considers the use of cable barriers undesirable for safety reasons and they are not used; however, their specific safety issues were not cited. Known examples of 2+1 passing lanes within the UK do not involve cable barriers, nor do those identified in France (see figure 3.1).

Figure 3.1 Undivided 2+1 passing lane in France (Glen Koorey)

There are a number of possible reasons why WRBs might not be preferred:

- Although generally good at preventing head-on crashes, the proximity of the barriers to the traffic lanes may lead to an increased number of minor crashes.
- For the single-lane section, particularly if it is constrained by a narrow cross-section and/or a roadside barrier, there might be the concern that vehicle breakdowns or crashes could block the road in that direction, with no ability to get past (also an issue for emergency services to get there).
- Regular encroachments of vehicles into the barrier could lead to an increase in maintenance costs to repair the barrier (and associated traffic disruption).

- The additional width required to install a WRB instead of just road markings might make it difficult to retrofit a 2+1 road on an existing wide two-lane carriageway.
- A barrier makes it impossible for traffic on the single-lane section to use the opposing lanes for overtaking (as is often done for a standalone passing lane), thus limiting its efficiency.

Sweden also developed a 2+1 road configuration that did not use a median WRB, instead using a painted median to divide traffic, and applying an upper speed limit of 90km/h. Carlsson (2009) reports that the painted median configuration did not achieve fatal and serious injury rate reductions as good as median WRB 2+1 roads (which achieved 50% reductions in fatal crashes); rather the reductions were in the order of only half that of the median WRB configurations.

Bergh et al (2005) identified the effects of 2+1 WRBs on different types of crashes in Sweden, compared with the previous wide two-lane roads:

- Head-on crashes were virtually eliminated, save for any driver going the wrong way on a section.
- Run-off road crashes were no different without additional roadside protection work.
- Overtaking crashes were reduced by ~45%.
- Rear-end crashes increased significantly, typically at least double.
- Intersection (turning off) crashes were reduced significantly by ~85%.

Of particular note was the significant reduction in fatalities, estimated in the long term to be ~75%, which was largely predicated by the prevention of head-on crashes. However, crashes with the barriers were not insignificant with an estimated rate of 0.5 crashes per million veh-km occurring over nearly 1,000km of 2+1 WRB roads. This translates into nearly two WRB crashes/year on average per km of road. These crashes were significantly reduced when Safe Systems practices were applied.

Interestingly, Bergh et al (2005) also found there were fewer barrier crashes in the transition zones (~5% of total WRB crashes) relative to their overall prevalence (10% to 12% of road length). Conversely, on the single-lane sections comprising 50% of the road length, the proportion of barrier crashes was ~65% to 70%.

Some commentators have expressed concern about the safety of WRBs for motorcyclists, due to the potential risk of a rider hitting the posts/cables at speed. Bergh et al (2005) found no evidence that the WRB created such a crash or worsened the consequences of any motorcyclist crashes on 2+1 roads in Sweden. With WRBs it is not possible to retrofit 'bikesafe' solutions such as sheet steel protection spanning z-posts to spread the force of impact. It is understood that motorcycle lobby groups in Europe have made progress against the use of WRBs on the basis of EU human rights legislation.

WRB has been reported to be very cost effective as a barrier system following application in Victoria, with Szwed (2002) recommending that a WRB be the preferred barrier system and alternatives only used where there is a constraint such as inadequate deflection clearances. The reported reduction in run-off-road crashes was 92%. While installation and maintenance costs were reported as competitive with alternatives, the impact on road trauma reduction was reported to make WRBs the most cost-effective system.

Table 3.3 summarises current advice on the installation of median style treatments in New Zealand (NZ Transport Agency 2011)

Table 3.3 Median treatments on rural roads

Traffic volume (vpd)	Treatment	Status
12,000–15,000	WRB or solid median	Recommended
8,000–15,000	WRB	To be considered
<8,000	ATP markings	To be considered
<5,000	Flush median	To be considered

As previously stated, the existing passing lane strategy indicates 2+1 roadways may be considered in a traffic volume band of 12,000vpd to 25,000vpd. Based on table 3.3, this suggests that all 2+1 roadways in New Zealand would fall within a traffic volume band where WRB medians are recommended.

There are examples of existing or committed WRB systems in New Zealand, including: SH1 Longswamp to Rangiriri (Waikato), SH2 Moonshine to Silverstream (Wellington) and SH1 Centennial Highway (Wellington). SH1 Longswamp to Rangiriri is an example of a 2+1 configuration, the other examples include some passing lanes and the safety implications are expected to be similar.

The following crash data is reported for the SH1 Longswamp to Rangiriri three-laning WRB project (Crowther and Swears 2010).

- 64% increase in crashes
- 63% reduction in fatal + serious crashes
- 51% reduction in injury crashes
- 25% reduction in the social cost of crashes.

In essence, the impact of the WRB has been the prevention of injury crashes at the expense of a greater number of non-injury and low severity crashes mainly through hitting the barrier (once every 10 days on average). As can be expected, the barrier effectively eliminated head-on crashes. A wide central median (providing at least barrier deflection) and better delineation were both post-construction recommendations to minimise the number of low severity crashes associated with WRB installation, as well as barrier maintenance. However, it is worthy of note that the Transport Agency's latest rural road strategy is aligned with an acceptance of road treatments resulting in an increased number of non-injury crashes if the same treatment results in a significant reduction of fatalities and serious injuries.

The crash data below is reported for the Centennial Highway project (Austroads 2009). This project also involved a speed reduction from 100km/h to 80km/h, which is likely to be attributable to a proportion of the crash reductions observed.

- 24% increase in crashes
- 100% reduction in fatal + serious crashes
- 70% reduction in injury crashes
- 99% reduction in the social cost of crashes.

The crash savings above show a similar pattern to those observed on the SH1 Longswamp to Rangiriri project, being an increase in overall crash numbers but a reduction in crash severity. In fact, there were no serious injury or fatality crashes within the five-year period since construction of the first section of the WRB in 2004.

Crash reductions associated with WRBs are not specifically identified within the EEM. The closest treatment for which crash reductions are specified is 'guardrailing', with the following crash benefits:

- 40% reduction in fatal crashes
- 30% reduction in serious injury crashes
- 10% reduction in minor injury crashes.

These crash savings are of a lower magnitude than observed on the SH1 Longswamp to Rangiriri WRB project, but are in keeping with the EEM's generally conservative estimates.

3.5 Adopted safety benefits

Based on available literature, a 25% typical injury crash reduction has been applied for this investigation over the length of the passing lane, as no apparent research has been undertaken subsequent to the current EEM which suggests variation to those procedures is justified. This crash reduction also compares well with European experience for 2+1 roadways. Although 2+1 roadways are likely to be installed on two-lane, two-way roads where volumes are nearing capacity, there is no literature that suggests the generic 25% reduction is invalid at these higher flow volumes.

Following the last passing lane in the study section, crash reductions are assumed to reduce linearly from 25% to zero over 5km, to account for downstream benefits as per the existing EEM procedures.

Where a WRB is installed as part of a 2+1 scheme a crash reduction of 50% is assumed, as observed on the SH1 Longswamp to Rangiriri WRB scheme. As the use of WRB avoids crossing-the-centreline crashes that could be attributed to overtaking, it is assumed that crash reductions from passing lanes would not be additional to those generated by a WRB where installed.

3.6 Safety summary

The design characteristics of the international 2+1 roadways schemes are summarised in table 3.4 in conjunction with the measured and estimated safety benefits where data is available.

Table 3.4 also provides details of two New Zealand WRB projects as well as the characteristics of the 2+1 roadway design proposed as part of this research.

Through a comparison of the traffic volumes, physical layout of the proposed 2+1 roadway and the changes in the different types of crashes, it is estimated that the introduction of 2+1 roadways in line with the proposed design, including a WRB, will result in the following safety characteristics:

- reduction in fatal crashes: 48%
- reduction in injury crashes: 33%
- increase in non-injury crashes: 44%.

The values identified above are highlighted in bold red in table 3.4.

Table 3.4 Summary of 2+1 design and safety characteristics

Country	WRB	Shoulder	Lane	Median	Total width	Length road	AADT	Total crashes	Fatal crashes	Fatal and serious crashes	Injury crashes	Comments
UK (design standard)	No	1.0	3.5	1.0	13.5	–	–	–	–	–	–	'WS2+1' design
UK (constructed)	No	3.5	3.5	0.75	11.5	–	–	–	–	–	–	
Germany	No	0.25	3.25–4.25	0.5	11–12	360km	15–25,000	–	–	–	-36%	
Finland	No	1.25	3.25–3.75	0.0	13.0	–	–	–	–	–	–	
Finland (theoretical)	No	–	–	–	–	48km	<14,000	–	0%	–	-11%	
Finland	Yes	0.9–1.25	3.25–3.75	1.7	14.35	–	–	–	–	–		
Finland (theoretical)	Yes	–	–	–	–	–	–	–	-46%	–	-25%	
Denmark	No	–	–	–	–	24km	7–15,000	–	-73%	–	-17%	
Sweden	Yes	0.5–1.0	3.25–3.75	1.25–1.75	13–14	950km	4–20,000	–	-50%	–	-40%	
US (theoretical)	No	–	–	–	–	–	–	–	–	–	-24%	
2+1 average with WRB	Yes	–	–	–	–	–	–	–	-48%	–	-33%	
2+1 average without WRB	No	–	–	–	–	–	–	–	-37%	–	-22%	
Design variation	Varies	0.25–3.5	3.25–4.25	0–1.75	11–14.35	–	–	–	0–73%	–	11%–40%	
Design proposal	Yes	1–1.5	3.5	2.0	15.0	–	>10,000	–	–	–	–	
New Zealand WRB schemes												
Longswamp to Rangiriri	Yes	–	–	–	–	9km	19,500	+64	–	-63%	-51%	
Centennial Highway	Yes	–	–	–	–	–	23,300	+24	–	-100%	-70%	Speed restriction implemented as well
New Zealand WRB average	Yes	–	–	–	–	–	–	+44%	–	-82%	-60%	

It should be noted that the New Zealand data in table 3.4 only represents a small sample and hence the average presented above may be misleading.

4 Proposed 2+1 design

Taking on board the previous consideration of design, safety and operational aspects of 2+1 installations around the world, the following section outlines a proposed design for New Zealand. However it is noted that the Transport Agency is currently reviewing cross-sectional standards and guidelines.

4.1 Design philosophy

Drivers are generally comfortable with the purpose, design and operation of passing lanes within New Zealand due to the quantity of passing lane treatments applied throughout the extent of the country. However customer surveys by the Transport Agency indicate that drivers want more passing opportunities.

On the basis that New Zealand's 2+1 roadways will essentially function as continuous alternating passing lanes, the proposed 2+1 roadway design incorporates the established passing lane design characteristics wherever possible.

There is significant research demonstrating the safety benefits for deploying WRBs with respect to the reduction of fatal and serious injury crashes. Although 2+1 layouts without WRB have also produced good crash reductions, options with WRB have generally had a far better safety performance. For this reason, WRB is considered a valuable component of 2+1 roadways especially at higher volumes. WRBs are considered particularly beneficial through critical transitions. Facilities to remove links and/or provide U-turn facilities/access should be considered on a site by site basis.

The proposed 2+1 roadway design is illustrated in figure 4.1. Some commentary about the design's characteristics is provided below. The design characteristics are within the extent of the ranges adopted by other countries, generally at the more conservative end (eg shoulder, lane and central median widths are generous).

4.1.1 Carriageway widths

All lanes should have a 3.5m width as per the *State highway geometric design manual* (SHGDM) (Transit NZ 2000) and MoTSaM.

The high traffic volumes expected at 2+1 road sections would not permit any reduction in width. 3.5m is highest standard (largest width) used on traffic lanes within New Zealand.

4.1.2 Central median

The basis for a 2.0m median incorporating a WRB is described broadly below. As noted with respect to WRBs, separate research is currently being conducted on the merits of other median widths.

Transport Agency experience is that a 1.5m WRB median leads to undesirably high maintenance costs from frequent (lower-severity and non-injury) barrier strikes. The Transport Agency is currently conducting research in relation to this issue. It is desirable to contain a deflection zone with the median. Deflections nominally range from 1.1m to 1.5m, but can be influenced by post spacing, the angle of hit, and the mass and speed of the vehicle hitting the WRB.

New Zealand guidance in Crowther and Swears (2010) is that WRB should be installed to minimise maintenance costs where practical, which means installing the barriers on a wide median to limit barrier collisions. 'Ideally, the median width should provide at least sufficient space to fully accommodate the design deflection of the selected barrier system: however this is not crucial given it is recognised that actual risk of a collision with an opposing vehicle is quite low even with a narrow median.' Barrier deflection systems may involve deflections of 1.1m to 1.5m (CSP Pacific 2012), suggesting a 2m to 3m median would ideally be installed in association with the WRB.

It is recommended that rumble strips are utilised on the median lane lines to further protect the barrier from vehicles wandering due to driver inattention.

4.1.3 Shoulder widths

Shoulder widths should be 1.5m for a single-lane section (as per the SHGDM for standard high-volume roads), and 1.0m on two-lane sections (as per the SHGDM for auxiliary lanes).

This arrangement creates a small but desirable 'wine bottle' effect at the merge. As reported by Crowther and Swears (2008) in their review of WRB from Longswamp to Rangiriri, the central median is widened giving extra width adjacent to the barrier enabling vehicles to make evasive manoeuvres as necessary and reduce the potential for WRB strikes in the vicinity of a merge.

It is noted however that MoTSaM suggests that normal shoulder widths should be continued unchanged through passing lanes, but Austroads (2010) is comfortable with a minimum of 1.0m next to auxiliary lanes. One concern with narrower shoulder widths is that loss-of-control crashes may be accentuated; recent research by Opus (2012) seems to confirm Koorey et al (1999), who found an increase in these types of crashes at passing lanes.

In conjunction with the carriageway and central median widths described above, this results in a total permissible width of the single lane section of 5.9m (assuming a 0.1m width for the WRB). In the event of a broken-down HCV (measuring 2.5m wide) having manoeuvred to the extents of the shoulder, this gives a width of 3.4m for onward vehicles to pass the stationary vehicle. It is also anticipated that some unsealed shoulder would exist beyond the extent of the 1.0m and 1.5m shoulders proposed for the design; generally unsealed shoulders are not encouraged on higher-volume state highways.

The proposed approach outlined above does not take into account any use of edge barriers.

4.1.4 Critical and non-critical transitions

Diverge (non-critical) taper – as per MoTSaM: 70m (70km/h) to 100m (100km/h).

Continuity line as specified within MoTSaM also proposed at diverge zone. No separation between tapers proposed.

Merge (critical) taper – as per MoTSaM: 115m (70km/h) to 160m (100km/h).

Separation between tapers: 80m, based on Austroads requirement for merging passing lanes to be separated by: taper in direction A + 3 sec of travel + taper in direction B.

The 80m separation between tapers increases the width of the median that provides the runoff zone that is provided by a wide shoulder in a normal passing lane design.

4.1.5 Signage

Use of the following signs in accordance with MoTSaM:

- RG22 – Keep left unless passing
- PW43 – Road narrows (right side narrowing);
- PW43.2 – Road narrows (right side narrowing) 200m ahead.

4.1.6 Road crown

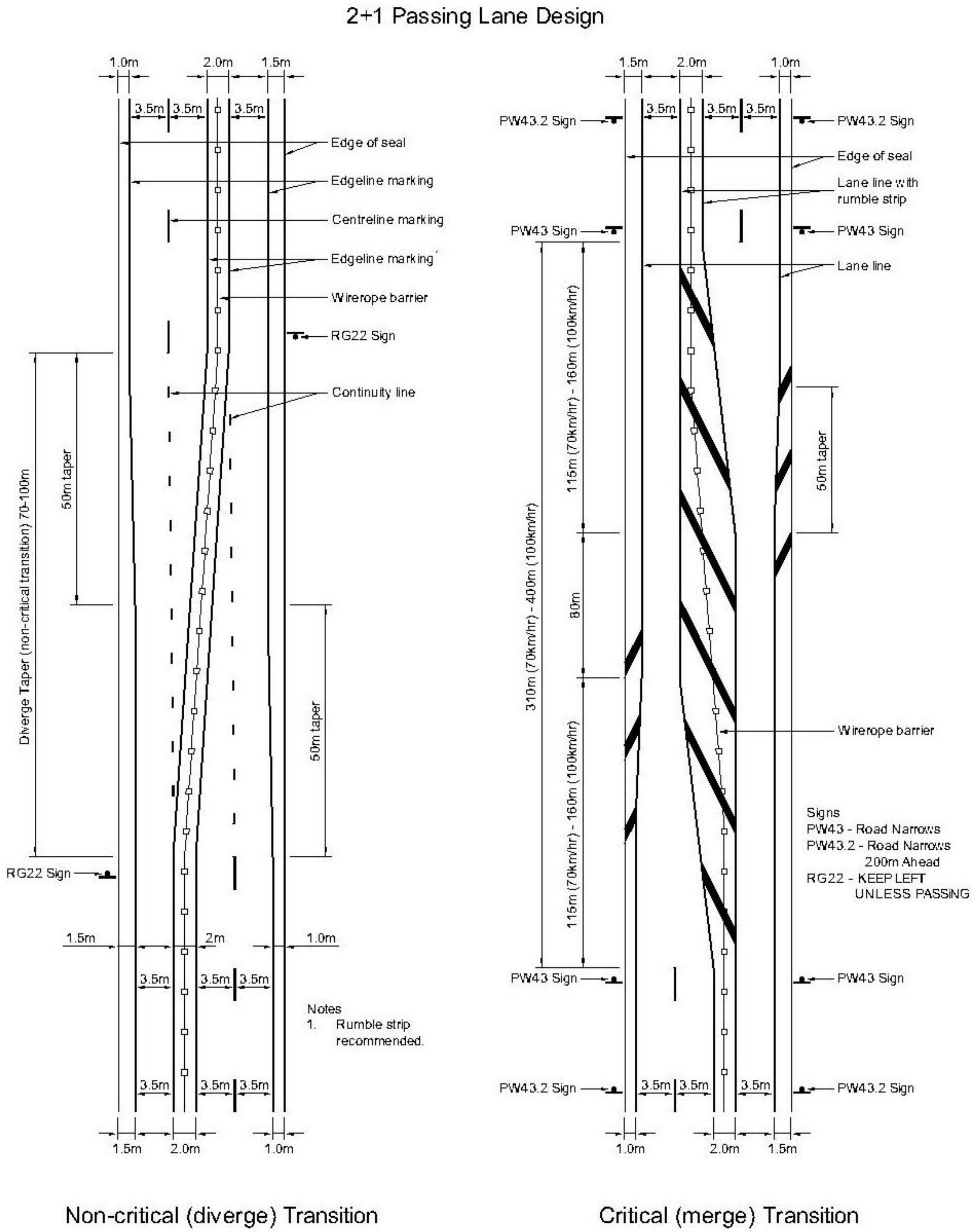
The research has suggested there are no particular advantages or disadvantages associated with the location of the crown from the specific perspective of a 2+1 roadway design (this issue is less important with tack-on passing lanes compared with 2+1 layouts). The proposed design has the crown running at the offside position of the overtaking lane. This position sees the crown 'zig-zagging' throughout the road section. Within a 15m roadway width, the crown will shift horizontally by 1m within each transition.

4.2 Proposed design

Based on an existing two-way two-lane highway with a 10m carriageway, upgrading to a 2+1 road with a WRB would involve up to a 16.5m carriageway. This assumes 1.5m wide shoulders are retained and that a 3m median is provided to house the WRB. This conservative cross section is unlikely to be adopted in New Zealand, and is significantly more conservative than overseas cross sections where 2+1 roads are achieved on carriageways as narrow as 11m. Shoulder widths of 1.0m to 1.5m and a central median of 2m would be more acceptable, being at the upper range of widths adopted overseas. This provides the cross section given in figure 4.1.

It should be noted that the road safety performance of 2+1 roadways and design layouts could be explored in more detail in relation to how this type of road configuration might support the Safe System principles. Further to this, it should be noted that the Transport Agency is currently reviewing design standards and guidelines including cross sections for passing facilities and safety treatment such as WRBs. This review is taking account of Safe System principles.

Figure 4.1 Proposed 2+1 roadway design



5 Establishing the operating characteristics

5.1 Overview

In order to establish the operating characteristics of moderate to high-volume passing lanes and 2+1 layouts (and to develop the detail required in the option evaluation process, see appendix A), a traffic modelling exercise was undertaken.

The foundation of the traffic modelling was based on data collected at passing lanes within New Zealand. Once the models were calibrated and validated, those core parameters were used to predict the operation of passing lanes and 2+1 lanes of varying length and spacings, AADT flow range, percentage of heavy vehicles and growth scenarios.

To ensure that the traffic models predicted the operation of moderate to high-volume passing lanes and 2+1 layouts accurately, they were built, calibrated and validated against a range of operational and behavioural characteristics as measured in the field. The key requirements for the passing lane field survey sites could be captured at three distinct sites as follows:

- site 1 – rolling/flat with low to moderate volumes
- site 2 – mountainous with moderate volumes
- site 3 – rolling/flat with high volume.

It was important that the sites also had sufficient, uninterrupted, upstream and downstream sections to ensure the model reflected vehicle platoon build-up and dispersion appropriately, including robust prediction of the benefits of the passing lane dissipating beyond the end of the passing lane and throughout the downstream single-lane section.

In order to identify the three suitable sites, the following activities were undertaken:

- review of NZ Transport Agency's 'Passing and overtaking road section details'[1], which identifies terrain type, estimated (2010) and projected (2020 and 2040) AADTs and section lengths for every passing lane in New Zealand
- review of previous research, with an emphasis on Cenek and Lester (2008) data
- review of detailed aerial photography through routes of interest
- review of state highway network-wide video footage (provided by the Transport Agency) as captured by the high-speed capture survey
- detailed interrogation of the Transport Agency's traffic monitoring system (TMS) data with a particular focus on traffic volumes and actual annual growth factors.

[1] Unpublished data collated by the Transport Agency.

The investigation quickly identified that very few sites met all the required criteria. Securing a high-volume site was the most restrictive as they were generally located on the fringe of New Zealand's major centres, where four-laning was already common.

The three sites chosen for field survey and their high-level characteristics were as follows:

- Site 1 – northbound passing pane north of Otaki, SH1. Terrain: flat/rolling, 2011 AADT: 13,930, HCV: 10.6% (taken from the Transport Agency TMS site ref: 01N00998), upstream section: 2.4km, downstream section: 5.8km, passing lane length: ~1km (surveyed during typically higher flow period).
- Site 2 – northbound passing lane, Haywards Hill, SH58. Terrain: mountainous, 2011 AADT: 13,760, HCV: 6% (taken from the Transport Agency TMS site ref: 05800000), upstream section: 1.1km, downstream section: 7.3km, passing lane length: ~1.2km.
- Site 3 – southbound passing lane, River Road, SH2. Terrain: flat, 2011 AADT: 23,720, HCV: 5.3% (taken from the Transport Agency TMS site ref: 00200957), upstream section: 2km, downstream section: 1km, passing lane length: ~750m.

Over and above the desktop analysis undertaken to identify the three sites, each of the sites were physically surveyed to ensure they were fit for purpose.

5.2 Field surveys

The field surveys were undertaken to enable a thorough understanding of the passing lane operation and driver behaviours. For each site, a 'study area' was defined. This was determined by an upstream and downstream 'feature', or constraint, which would otherwise potentially change driver behaviours. For each site, the constraints are shown in table 5.1.

Table 5.1 Study area upstream and downstream constraints

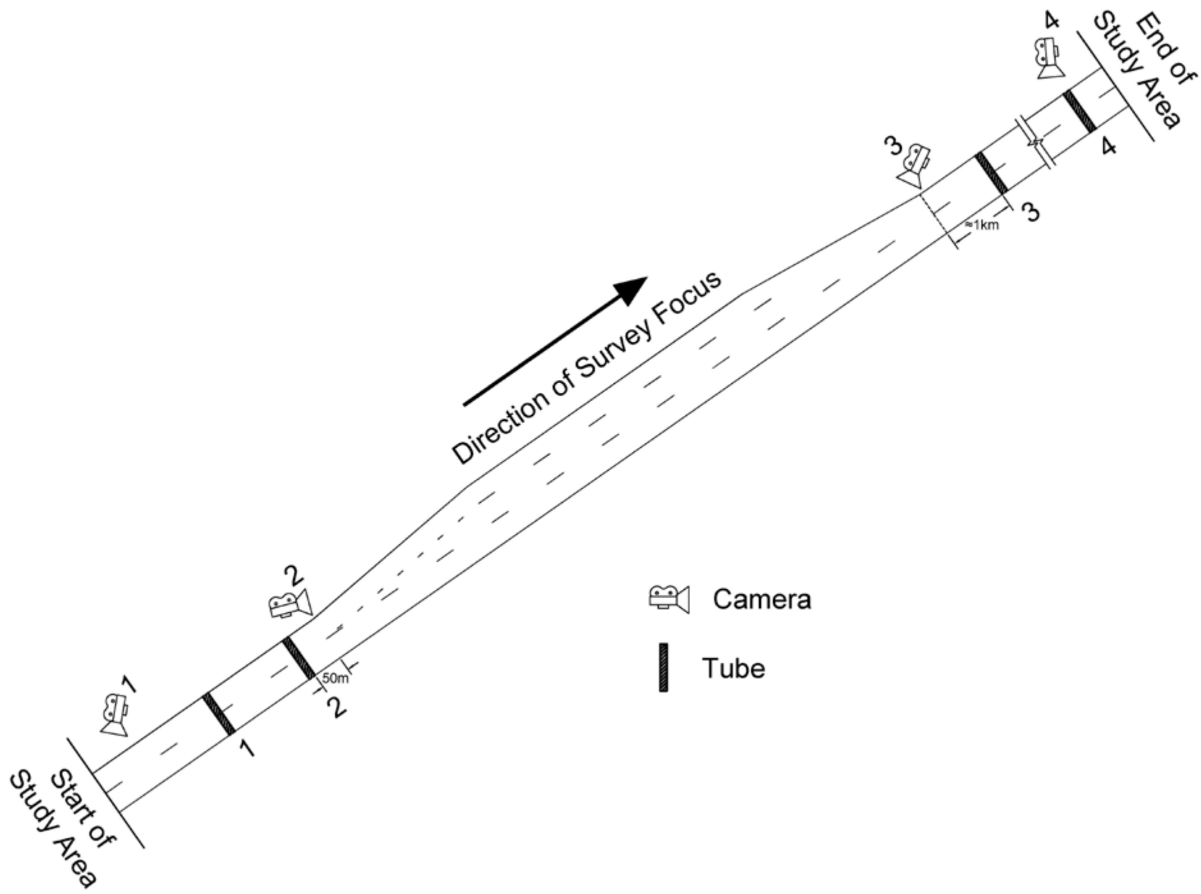
Site	Upstream	Downstream
Site 1 - Otaki	50/100km/h speed restriction	Next northbound passing lane
Site 2 - Haywards Hill	Intersection with Hebden Crescent	Roundabout upstream of Pauatahanui
Site 3 - River Road	Moonshine bridge	Signals at intersection with Fergusson Drive

The field surveys were all carried out in fine, dry weather, and the operation of the passing lane was considered to be typical at each site (ie there were no abnormal events during the survey periods). The Otaki field surveys were carried out on a Friday to capture the higher traffic volumes. The traffic patterns at Haywards Hill and River Road were more consistent and, as such, the field surveys were undertaken mid-week.

The following system was used to collect data at each site. This is illustrated in four tube counters:

- four automatic number plate recognition (ANPR) cameras
- two video cameras to record merge and diverge activity and on-road behaviour located at the beginning and end of the passing lane only
- floating car GPS surveys through route (25 to 32 runs in direction of PL during survey day).

Figure 5.1 Passing lane field survey schematic



5.3 Survey data/model calibration and validation parameters

The survey data, in conjunction with data from existing sources, has been used in the traffic models to establish and forecast the operating characteristics of a range of scenarios. Table 5.2 summarises the input requirements for the traffic models and the corresponding data source used to establish those characteristics. Details of the traffic modelling are described in the next chapter.

Table 5.2 Data sources

Characteristic	Data source
Geometric layout	Correct scaled aerial photography imported into model background
	Vertical profile
On-road behaviour	
• Passing manoeuvres	• ANPR cameras
• Tendency for passing lane preference by vehicle type	• Video cameras and GPS floating car
• Merging and tendency for later merging	• Video cameras and tube loop counters
• Effect of gradients	• Video cameras and GPS floating car survey
• Local environment and conditions affecting traffic flow and driver behaviour	• Tube counters, TMS system, video cameras
Classified traffic flow, 15min intervals	Tube counters and NZ Transport Agency TMS system
Speed distribution through study area	Tube loop counters
Headway distribution through study area	Tube loop counters
% overtaking in study area sections	ANPR cameras
Travel time (and average speed) by distance segment through study area	GPS floating car

6 Microsimulation modelling

6.1 Microsimulation modelling software

Microsimulation, as established through previous research, is the only traffic modelling tool capable of realistically evaluating the travel time savings of passing lanes. This is due to its abilities to represent platoon formation upstream of the passing lane, driver behaviour and desired speed through overtaking areas, opportunistic overtaking on the opposite carriageway, and the travel time savings that accumulate downstream of the passing lane as platoons disperse. Platoon formation, dispersal and opposite carriageway overtaking cannot be realistically represented in a deterministic model.

S-Paramics microsimulation traffic software suite was used to undertake this study. There were a number of reasons for this, the most notable being:

- the software's historically established ability to represent vehicle speed distributions and predict journey times under various flow and heavy vehicle scenarios on rural New Zealand state highways
- the ability to link the software directly with the preferred ITS-assisted merge design strategy
- the application of S-Paramics to previous research projects of this nature and the strength of the outcomes of this work, eg establishment of optimal length and spacing for UK Trunk Road WS2+1 overtaking arrangements leading to the development and ratification of economic procedures for assessing passing lanes in Highways Agency (2008, vol 6, section 1 'Design of WS2+1 roads').

In terms of general functionality, S-Paramics has the added benefit over comparable microsimulation packages of being able to develop 12- to 24-hour demand matrices with flow profiles controlling the build-up and dissipation of demand throughout the day. 12+ hour matrices will be required to annualise operational outputs and evaluate economic benefits. S-Paramics management of traffic flow profiles is superior to that of most other traffic modelling software that uses hourly flow rates, which then have to be combined later.

S-Paramics microsimulation also provides a wealth of output data such as journey time reliability, vehicle headways and vehicle proximity outputs. These outputs have been used to assist in the determination of driver frustration and safety benefits.

6.2 Modelling methodology overview

The traffic modelling process followed a logical approach to ensure that the observed behaviours and characteristics from the field surveys were present in each of the scenario tests.

The first task was to create separate traffic models to reflect the geometry and traffic characteristics (model inputs) of each of the three survey sites. The models used classified 15 minute flow data to develop traffic demands and flow profiles travelling through each study area and was coded against detailed aerial photography backgrounds. A 24-hour model was developed for each site, disaggregated by AM peak, interpeak and PM peak, using six vehicle types (based on the Transport Agency's classification system).

Initially a small number of core model parameters (notably global headway factor and end-of-passing lane ahead warning distance) were calibrated for site 1. These parameters were used in site 2 and site 3, with little to no modification required to produce one set of 'typical' parameters which delivered robust calibration/validation results across all three sites. The result of this process enabled the identification of a set of 'core' parameters; one with a sub-set variation for i) flat/rolling terrain and ii) mountainous terrain. These core parameters were then used in a generic model to establish the impacts of varying terrain, traffic volumes, passing lane length and spacing, percentage of HCVs and forecast growth scenarios. The differences in each of the model outputs, (travel time savings, vehicle operating costs (VOCs), platooning levels) were used to inform the economic evaluation.

The traffic models were successfully calibrated and validated to represent the operation and behaviours observed during the field surveys. Sections 6.3 and 6.4 summarise this process.

6.3 Traffic model calibration

6.3.1 Overview

The traffic models were separately calibrated to three key observed data, for the AM, IP and PM periods as follows:

- speed distribution
- headway distribution
- % overtaking, focused on the % of overtaking in the passing lane.

For both speed and headway, an important aspect of the calibration was to replicate the distribution of on-road behaviours within the model, ie the range and shape of the distribution. This is because the model needs to represent, to an appropriate level, the individual events that produce the aggregate outputs (such as total journey time), for example:

- passing manoeuvres completed by vehicles wishing to travel a certain degree faster than vehicles in front
- formation of platoons due to faster vehicles catching up with slower vehicles.

Once the model was calibrated across 4 loops x 3 time periods x 3 sites to replicate these on-road behaviours, it was used to predict the change in aggregate outcome (average journey time, average speed) based on varying inputs such as traffic volume/characteristics and on-road geometry in the scenario testing phase of the project. The model parameters used for the calibration were taken forward and used in the generic models

6.3.2 Speed calibration

The difference between the observed and modelled average speed at each loop, for each site, for each time period was calculated. From this set of comparisons, the number falling below a certain threshold (eg modelled speeds within 5% of observed) was determined and the results expressed as a percentage of the total number of comparisons (12 for each time period, 3 sites and 4 loops). The results are summarised in table 6.1.

Table 6.1 Summary of observed and modelled average speed comparison

Observed speed threshold	AM	IP	PM
<2.5%	50%	33%	42%
<5%	83%	75%	75%
<10%	100%	92%	100%
Average	**0.5%**	**2.4%**	**-1.0%**

The average presented in the bottom line of table 6.1 is the average of the differences between each of the observed and modelled values.

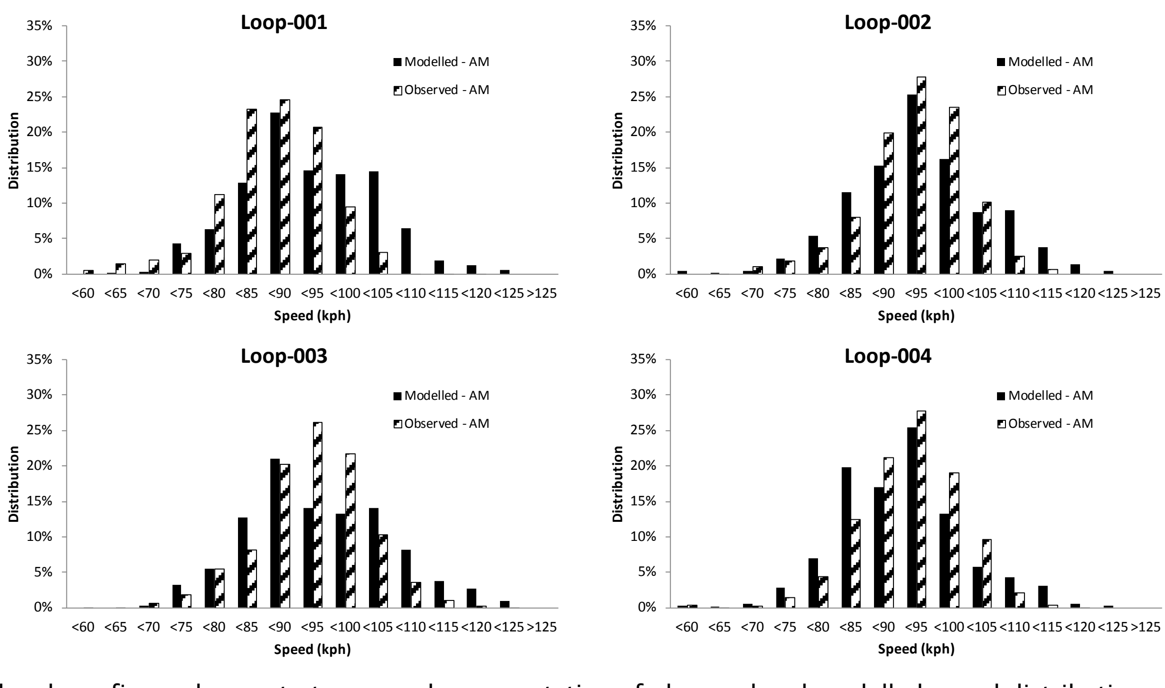
The speeds in the majority of sites compare well with the observed average, with a notable proportion of locations being within 2.5% of the observed average.

At each loop, and each site, minor site specific conditions will result in varying observed speeds. The average difference has been presented to demonstrate that there is no systematic under or over estimates of speeds within the model. This average compares well with the complete set of observed speed data.

As noted above, the distribution of speeds is important to represent in the model. As an example, the speed distribution for a single period for the Otaki site has been presented in figure 6.1.

It is important to note that significance should not be placed on the absolute difference between individual comparisons (bars) within each graph. The distribution graphs are based on a bin analysis, ie a count of the samples that fall within certain ranges (bins). The individual bars for the observed and modelled samples may change from step to step depending on how the bins are defined. The key component is the shape of the distribution, and more notably, whether the modelled distribution shape changes through the study area in a similar pattern to what is observed.

Figure 6.1 Example speed distribution comparison – SH1 Otaki AM period



The above figure demonstrates a good representation of observed and modelled speed distributions, ie the overall shape/form, central tendency and spread (extents) of the datasets. Importantly, the change in distribution due to the passing lane and local geometry through the Otaki SH1 study area in the AM period is well reflected in the modelled data.

6.3.3 Percentage passing

The focus of the passing calibration was the percentage of overtaking occurring through the passing lane. A 'passing' or 'overtaking' manoeuvre was defined based on the timestamp for a vehicle entering and exiting the passing lane, recorded by the ANPR camera. A vehicle that exited the passing lane before a vehicle it was following at the start of the passing lane was counted as one manoeuvre, eg a vehicle that overtook three others in the passing lane produces three 'passes'. The observed and modelled rates have been compared for the passing lane section for each of the sites and the difference calculated, as shown in table 6.2.

Table 6.2 Difference between observed and modelled passing % through PL

Observed speed threshold	AM	Interpeak	PM
SH1	-1%	4%	-2%
SH2	6%	1%	-1%
SH58	-6%	-8%	-8%

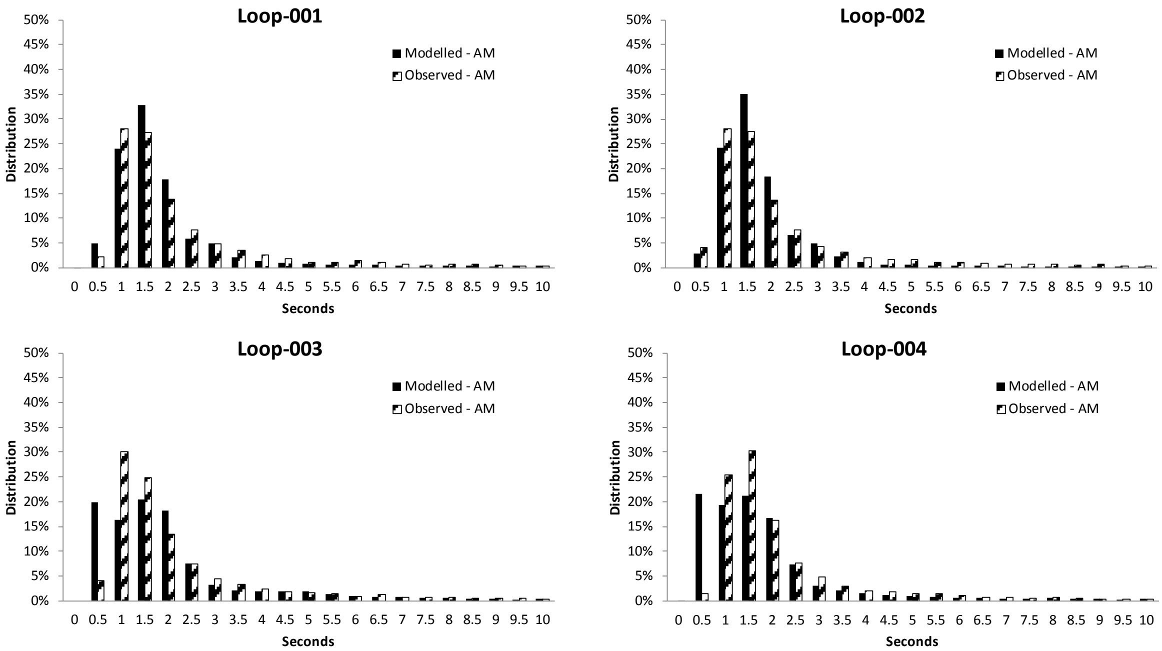
The data shows a good correlation between the observed and modelled passing percentages at SH1 and SH2 through all periods. The modelled data also demonstrated a strong correlation with the observed trend in the changes of passing percentage as volume increased/decreased, for example reducing passing percentages at SH2 during very high flow periods. High rates of passing (around 40% to 50%) were observed on SH58 Haywards Hill. The mountainous model method does not quite replicate these levels and may be conservative in the measurement of these passing lane benefits. The commuter traffic prevalent on SH58 may be more familiar with the route and hence more inclined to overtake; this may not be the case on other more remote mountainous routes.

Little to no overtaking was observed in the sections before and after the passing lane, generally less than 2% in all study areas. Overtaking in the face of on-coming traffic was enabled within the generic S-Paramics model, and this was checked to verify against the field data at around 2%.

6.3.4 Headway

Average headways cannot be compared due to a number of factors including difficulty in establishing a direct comparative value between observed and modelled data, miscellaneous headway recordings, the relatively small magnitude of average headways, and the wide variance at sites (eg from one second to hundreds of seconds). Therefore a sample distribution for a single time period, at each of the three sites was used. An example headway distribution for a single period for the SH2 River Road site is presented in figure 6.2. Again, the significance of the data is in the shape of the distribution and not the absolute values.

Figure 6.2 AM headway distribution – SH2 River Road



The distribution of modelled headways reflects well the changing characteristics of traffic platooning through each study area, the traffic volumes, on-road geometry and notably the influence on the passing lane.

6.4 Traffic model validation

The total travel time through the full length of the study area was the key aggregate output from the traffic modelling for assessing the optimal length and spacing arrangements, and to a lesser extent, the performance of the ITS-assisted merge solution. This output provided the measure of the scheme travel time savings, a large contributor to economic benefits.

The models were validated against the observed journey times through sections of the network. This was carried out as strict validation, ie an independent check, no alteration to the model parameters, geometry, etc was made to improve or alter this comparison. The observed versus modelled journey times for each site, across each time period, is shown in tables 6.3 to 6.5.

Table 6.3 Observed vs modelled study area journey times – SH1 Otaki

Period	Description	Observed (minutes: seconds)	Modelled (minutes: seconds)	Difference		Pass
				Sec	%	
AM	Northbound	06:42	06:36	-6	-1%	Yes
Interpeak	Northbound	06:42	06:46	4	1%	Yes
PM	Northbound	06:28	06:55	27	7%	Yes

Table 6.4 Observed vs modelled study area journey times – SH58 Haywards Hill

Period	Description	Observed (minutes: seconds)	Modelled (minutes: seconds)	Difference		Pass
				Sec	%	
AM	Northbound	06:29	06:30	1	0%	Yes
Interpeak	Northbound	06:08	06:16	8	2%	Yes
PM	Northbound	06:27	06:35	8	2%	Yes

Table 6.5 Observed vs modelled study area journey times – SH2 River Road

Period	Description	Observed (minutes : seconds)	Modelled (minutes : seconds)	Difference		Pass
				Sec	%	
AM	Southbound	02:28	02:40	12	8%	Yes
Interpeak	Southbound	02:28	02:27	-1	0%	Yes
PM	Southbound	02:10	02:31	21	16%	Yes

As indicated in the right-hand column of each of the above tables, the model meets industry best practice criteria (within 15% or one minute) as required by the New Zealand Modelling User Group[2] for category E models.

6.5 The generic models

The development, calibration and validation of the three existing test sites were used to confirm model parameters representing the New Zealand rural state highway environment and passing lane configuration and operation. This formed the basis of two generic models; one for flat/rolling terrain and one for mountainous terrain, both featuring the same 'core' parameters.

The use of a generic model ensured the model results are comparable, quantifiable and robust, eg that they are not influenced by any on-road constraints such as horizontal/vertical geometry present in existing site locations. The generic modelling was used to assess and understand the effect of following scenarios:

- optimal length of passing lane facilities
- optimal spacing of successive downstream passing lanes
- traffic flow volume variations
- percentage of HCVs
- economic scheme life based on forecasted traffic growth
- the operation and effectiveness of the ITS-merging applications.

[2] www.ipenz.org.nz/ipenztg/Subgroups/NZMUGS/Documents/130412-NZMUGS-Draft-Observed-vs-Modelling-Comparison-Criteria.pdf

6.6 Generic model parameters and setup

The following sections cover the generic model parameters and setup features, developed from the data collection and model calibration completed at the three survey sites. These generic models (and associated settings) were used to derive economic outputs.

The EEM defines road types in terms of vertical terrain as flat, rolling, hilly and mountainous and in terms of horizontal terrain as straight, curved, winding and tortuous. For the purpose of this research investigation, two generic model types were developed:

- generic model 1: flat – rolling (straight – curved)
- generic model 2: hilly – mountainous (winding).

The characteristics and differences of the two generic models are described in the following sections.

6.6.1 Generic model 1: flat – rolling terrain

- **Headway:** Factor of 1.5 (default 1.0) used to increase modelled headways to represent observed headway distributions.
- **Modelled time-step (calculation interval):** Time-step of 3 (recalculating 3 times per second) was applied instead of the default 2, to improve visual representation of driver evaluation of following distance
- **Vehicle types:** Six vehicle types defined from the Transport Agency classification system: car, light towing, public transport vehicle, medium goods vehicle, large articulated and B-train.
- **Physical vehicle parameters:** Top speed, accelerations, size and weight developed from Transport Agency TMS vehicle classification data, observed data collected from three survey sites, and anecdotal observation of on-road operation and vehicle behaviour.
- **Vehicle gradient parameters:** Heavy vehicle acceleration reduction and top speed limiting defined for all except car vehicle types. Drag and inertia was set to zero for all vehicle types to remove double impact of gradient response parameters.
- **Gradient:** The gradient parameter within S-Paramics represents the impacts on vehicle top speeds and acceleration rates due to on-road gradients. Some interpolation between the on-road gradient and the S-Paramics parameter was required. At lower gradients, no significant factoring was required, but as gradients increased past 4% to 5% a proportionate increase was required to produce the observed speed and acceleration levels. The average S-Paramics gradients of approximately 1.2%, and maximum gradients of approximately 4%, are representative of the flat/rolling sites surveyed and fit with EEM definitions (table A7.5)
- **Link speeds:** S-Paramics contains two link types, urban and highway. Urban links represent typical behaviour on approach to urban-style intersections. Highway link types are more representative of open road and motorway environments. Urban link types have a narrow speed distribution and average speed similar to the limit specified. Highway link types have a wider speed distribution and produce average speeds higher than the specified limit. S-Paramics highway link type with speed 85km/h represents 100km/h New Zealand highway environment and the passing lane environment is

represented with a highway link speed of 90km/h. These speeds were calibrated to produce the speed distributions observed at the three survey sites across a range of traffic flow conditions.

- **Start of passing lane:** 200m of lead-in link length was applied before the physical start of the passing lane to represent the observed passing patterns at the start of the passing lane.
- **Speed lead-In:** 90km/h speed link was applied 100m prior to the start of the passing lane to represent the observed aggressiveness of drivers on the immediate approach to the passing lane. 85km/h speed was applied after the end of the passing lane. Vehicles travelling at high speeds at the end of the passing lane (eg vehicles having performed a passing manoeuvre) would continue to travel at higher speeds for some distance past the end of the passing lane. The median speed was higher than the link speed setting noted here, ie a modelled highway link speed of 85km/h has a median speed of around 92km/h to 95km/h dependent on geometry and traffic composition.
- **End of passing lane ahead warning:** The on-road standard warning distance was 250m. A modelled ‘signposting’ length of 500m was set at the end of the passing lane to inform drivers of the nearing end of the passing lane. This value reflected the observed merging behaviour and percentage passing levels with the PL.
- **Lane preference:** The passing lane was modelled as a lane gain to the right, so vehicles passing needed to move into the central (passing) lane to perform a passing manoeuvre (as signposted on-road).
- **Vehicle lane use:** No restrictions were placed on either lane, ie no restriction to heavy vehicles using the passing lane.
- **Opposite carriageway overtaking:** Very low rates of overtaking between passing lanes were observed on-road. This research investigated passing lanes in sequence (including 2+1 roadways) which are features of both the Otaki and River Road sites. Forewarning drivers of upcoming/regular passing lanes is known to reduce opposite carriageway overtaking before or after the passing lane. Opposite carriageway in the ‘with PL’ scenario was disabled.
- **Opposite carriageway overtaking (do minimum):** The do minimum scenario does not include any passing lanes and an overtaking rate of 2% to 3% was assumed based on historical research. The model was configured so that the 2% to 3% of opportunistic overtaking would only be permitted when opposing traffic volumes contained sufficient gaps for vehicles to perform this manoeuvre (as would occur on road).

6.6.2 Generic model 2: hilly – mountainous terrain

The points below note the differences between the above flat-rolling generic model setup and the hilly-mountainous generic model setup.

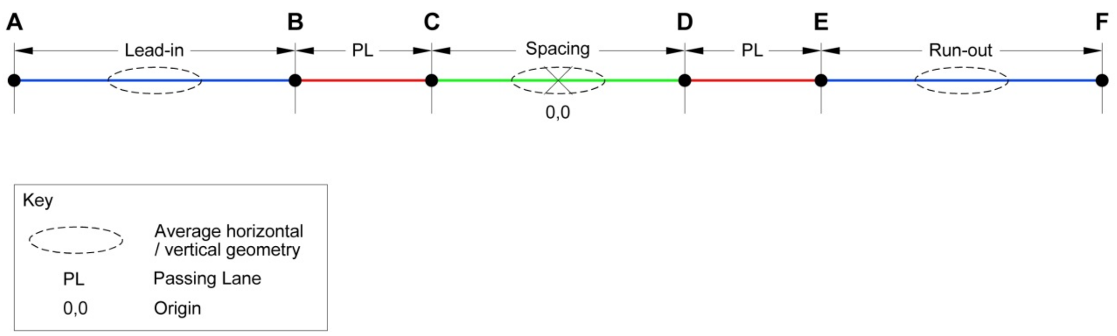
- **Gradient:** The average gradient used was approximately 3%, the maximum gradient used was approximately 10%. Higher gradients were applied over approximately 8% of the full length of the mountainous study area. This range of gradients is typical of hilly and mountainous terrains as per EEM definitions (table A7.5).
- **Link speeds:** Between passing lanes were set to a speed limit of 80km/h, passing lane links were set to a speed limit of 85km/h. Median speeds were higher, these values representing the distributions observed at the mountainous survey sites.

- **Lane preference:** The flat-rolling model assumed that vehicles preferred lane 1 (kerbside) and had to move into lane 2 to perform passing manoeuvres. To represent passing rates on mountainous sites the light vehicles were configured to prefer lane 2 (the passing lane) by default and the heavy vehicles lane 1.
- **Vehicle lane use:** Lane 2 (overtaking lane) was barred to all vehicles weighing over 2 tonnes. This meant that cars were the only vehicles permitted to use this lane and was based on observation of vehicle behaviour through the Haywards Hill study area and anecdotal evidence of climbing lane operation.
- **Start of passing lane:** Due to the differences noted above, no lead-in additional length to the passing lane was required and no lead-in speed difference was required.
- **End of passing lane ahead warning:** Signposting length of 250m was set at the end of the passing lane to inform drivers of the nearing end of the passing lane.

6.7 Physical generic model layout

The physical layout of the generic model is described diagrammatically in figure 6.3.

Figure 6.3 Physical layout of generic test model



Two simulated models were used to calculate economic outputs, a do minimum situation that did not include passing lanes B–C and D–E, and an ‘option model’ that included a passing lane in these locations.

The outputs from the model were used to derive the economic measures of travel time relating to savings in time spent travelling, driver frustration relating to time spent following, and VOCs relating to distance/gradients travelled and fuel consumption, as described later in section 8.4.

The physical layouts of the sections of the generic model were determined by examining typical/average rural state highway sections throughout New Zealand. This is described in the following sections. Some minor iteration and adjustment to these settings was required through the creation of the test models and generation of preliminary results.

6.7.1 Flat – rolling terrain

Total length: The total length of model (A to F) was fixed within each AADT band so that the travel times and travel distances would be comparable between the do minimum and all option model tests.

Lead-in/run-out length: This length was determined by measuring the existing passing lane spacing and distance from adjacent network features for a number of stretches of rural state highway corridor, and the desired spacing of facilities based on previous research. These distances were fixed for each AADT level tested as per the lengths shown in table 6.6. The distances chosen were mildly conservative so as to prevent unrealistically high levels of platoon development and as a result, overestimation of benefits.

Table 6.6 Passing lane spacing

AADT	On-road measured/ desired spacing (km)	Lead-in/run-out length (km)	Max spacing. Central area (km)
5,000	8.0 – 10.0	11.0	12.0
10,000	5.0 – 8.0	7.0	10.0
15,000	2.6 – 6.0	5.0	6.0
20,000		4.0	5.0
25,000		3.0	3.5

- **Effective option test length:** The effective length of the test option (B–E) is the lead-in length, plus the maximum length of passing lane (2 × 1.75km) plus the maximum spacing, plus the run-out length. As passing lane and spacing lengths are reduced/increased the location of the passing lane test length will shrink/expand around the origin, ie point B stays the same, but points C, D and E will move in/out. Points A and F will also remain at the same location to keep a consistent overall study length (and hence measure comparative travel times, distances, fuel use etc).
- **Average vertical terrain:** The average geometry sections featured vertical gradients of 1.5% to 4.5% across a 1km section. These values were developed through a review of data from a number of existing state highway sections around the country across a range of AADT levels and were consistent with EEM ranges given in table A7.5.
- **Average horizontal terrain:** The three average geometry sections featured two horizontal curves with speed advisory signs of 85km/h. The curve speeds were derived through a review of data from a number of existing state highway sections around the country across a range of AADT levels and were typical of straight to curved terrain as per EEM table A7.5.

Similar to lead-in lengths, the horizontal and vertical geometry settings were developed to be mildly conservative so as to prevent unrealistically high levels of platoon development and as a result, overestimation of benefits.

6.7.2 Hilly – mountainous terrain

The points below note the differences between the above flat-rolling physical model layout and the hilly-mountainous physical model layout.

- **Average vertical terrain:** The average geometry sections feature vertical gradients of 3.5% to 9% across a 1km section. These values were developed through a review of data from a number of existing hilly and mountainous state highway sections and are consistent with EEM ranges given in table A7.5.
- **Average horizontal terrain:** The average geometry sections feature two horizontal curves with speed advisory signs of 65km/h, and two horizontal curves with speed advisory signs of 45km/h. This is typical of winding terrain as per EEM table A7.5.

6.8 Traffic demand tests

The generic models require traffic flow (demand) inputs along with the physical description as described above. The demand inputs for each test model are described below.

6.8.1 Flat – rolling terrain

- **AADT (variable parameter):** Five scenarios were produced for AADT flow ranges with 5,000vpd increments starting with 5,000 through to 25,000.
- **Day type:** A seven-day average was represented by selecting typical state highway TMS site locations for each AADT flow range.
- **Time period and profile:** A 24-hour model was run for each scenario. The profile was determined from the TMS site data above for each AADT flow range. This demand data was produced for each vehicle type, in each travel direction.

The above points reduce the level of annualisation factoring for economic calculations. Seasonal variations in traffic flow were not considered. Traffic demands included the following refinements:

- **Vehicle classification (variable parameter):** The vehicle type composition was determined from the TMS site data above for each AADT flow range. The total heavy vehicle type percentage composition was varied to measure the impact of HCV composition.
- **Growth rate:** 0, +10, +20 and +30 year demand scenarios were created based on growth rates calculated from state highway traffic count data for light and heavy vehicle types. Growth rates were calculated separately for each AADT range.

6.8.2 Hilly – mountainous terrain

The points below note the differences between the above flat/rolling and the mountainous demand tests.

AADT (variable parameter): Three scenarios were produced for AADT flow ranges with 5,000vpd increments starting with 5,000 through to 15,000.

6.9 Option model summary

The previous sections above describe the generic model characteristics and the variables required to be tested to inform the economic evaluation. Across the tests undertaken, the following range of variables was assessed within the flat-rolling and/or hilly-mountainous models:

- AADT: 5,000 to 25,000vpd
- passing lane length: 500m to 1750m
- passing lane spacing: 1,250m to 19,400m
- HCV component (in year 0): 8% to 27%
- forecasts: year 0, year 10, year 20 and year 30.

7 ITS-assisted merging

7.1 Assisted merge concepts

The concept of assisted merging stems from the USA whereby traffic control techniques are employed at construction work zones to enable a greater volume of traffic through lane merges on multi-lane highways (Hallmark et al 2011). The techniques seek to reduce the impacts of merge conflicts, improve vehicle merging behaviour, reduce driver frustration and maximise queue storage capacity. This is achieved through two techniques:

- **Early merge:** whereby motorists are encouraged to merge upstream of the physical merge so the merge itself can deliver a more stable, higher-volume throughput.
- **Late merge:** whereby motorists are encouraged to make use of all available lanes and merge in an orderly fashion (ie take it in turns) at the physical merge. The late merge concept legitimises the use of all lanes during periods of congestion and reduces 'forced' merges, which create sudden vehicle braking and reduce traffic flow.

The traffic control techniques are communicated to motorists through various messages used on various types of roadside signs/infrastructure. Within the categories of both types of merge, there are two concepts that communicate to drivers how they should drive/behave on the approach to construction work zones:

- **Static (early/late) merge:** utilising a selection of fixed-plate, static signs that remain in place on a full-time basis.
- **Dynamic (early/late) merge:** utilising electrical signs and systems to detect and sign when particular conditions exist on-road (eg traffic volume at a particular location exceeds a particular threshold).

Construction work zones are temporary in nature and the use of static signing in these environments is particularly acceptable. This is not the case for the New Zealand passing lane environment, whereby the passing facility has been designed to operate in a particular way on a permanent basis for many years. For this reason, static merging was not considered any further within this research. The concept of dynamic merging, however, is more appropriate; activating traffic control systems to manage vehicles when passing lane volumes, flow and/or occupancy reach a certain level.

Computer simulation and field observations by Beacher et al (2004) considered 3-to-1, 3-to-2 and 2-to-1 large merges. Using the late merge concept, Beacher found that statistically significant results were only positive for the 3-to-1 merge. For the 3-to-2 and 2-to-1 merges, the merge capacity increased slightly only when the percentage of heavy vehicles exceeded 20%.

Beca (2010) undertook a literature survey and VISSIM microsimulation study to investigate the potential of ITS-assisted merging in the New Zealand context.

Using traffic flow demands containing 10% heavy vehicles, the traffic model was used to determine the potential benefits of ITS-assisted merge concepts as measured against a do nothing scenario. The flow rates and traffic speeds immediately downstream of the merge, in conjunction with travel times (taken from a point 100m upstream from the merge to the end of the simulation model) were the main evaluation criteria. A series of scenarios were modelled including varying the early merge 'location', use of speed restrictions and closing the passing lane. The concepts were understood and optimised using modelling techniques and sensitivity tests.

The results stated that the merge capacity of the do nothing scenario (ie no ITS-assisted merge solution) was 1,350vph (one direction). The merge capacity increased to 1,550vph and 1,520vph using early and late ITS-assisted merging techniques respectively. The traffic model used within the assessment was calibrated to mean speeds as identified in Bennett (1985). Vehicle headways were assessed against cumulative distributions at selected locations and flow rates.

With anecdotal reports that the Transport Agency was to close passing lanes when volumes were anticipated to be above 1,200vph (long weekends, events and holiday periods), the Beca analysis indicated that the ITS-assisted merge had potential to increase the traffic flow throughput by 320 to 350vph.

Section 7.2 below takes Beca research to the next investigation stage by utilising a purpose-built, calibrated and validated model to a range of New Zealand site survey data, including high-volume passing lanes, to assess the potential for ITS-assisted merging within New Zealand. The assessment process is described, with results clearly shown and discussed.

It is worth noting that the rural passing lane operation is quite different from that of American highway work construction zones. Furthermore, it is also quite different from rural 2-to-1 merges at the end of four-laning sections, such as the one on SH1 at Pukerua Bay. The decision to enter the finite length passing lane, perform some overtaking manoeuvre(s) and return to the nearside lane represent quite unique behaviours that are not really produced outside of the passing lane environment.

7.2 Merge testing and model configuration

In order to understand the value of a potential ITS-assisted merge application, it was first necessary to use the observed traffic data and model to understand how a passing lane merge operates in an uncontrolled manner (ie with no ITS-assisted merging in operation).

The merge tests were conducted using a separate model (ie different from the two generic models described above), which was based on the flat-rolling generic model but considerably shorter in length. The model used a 3km lead-in section, a 1,200m passing lane and a 3.1km lead-out section and ran for a 24-hour period. An arbitrary high-volume traffic demand, increasing above 1,300vph in one direction, was fed into the model.

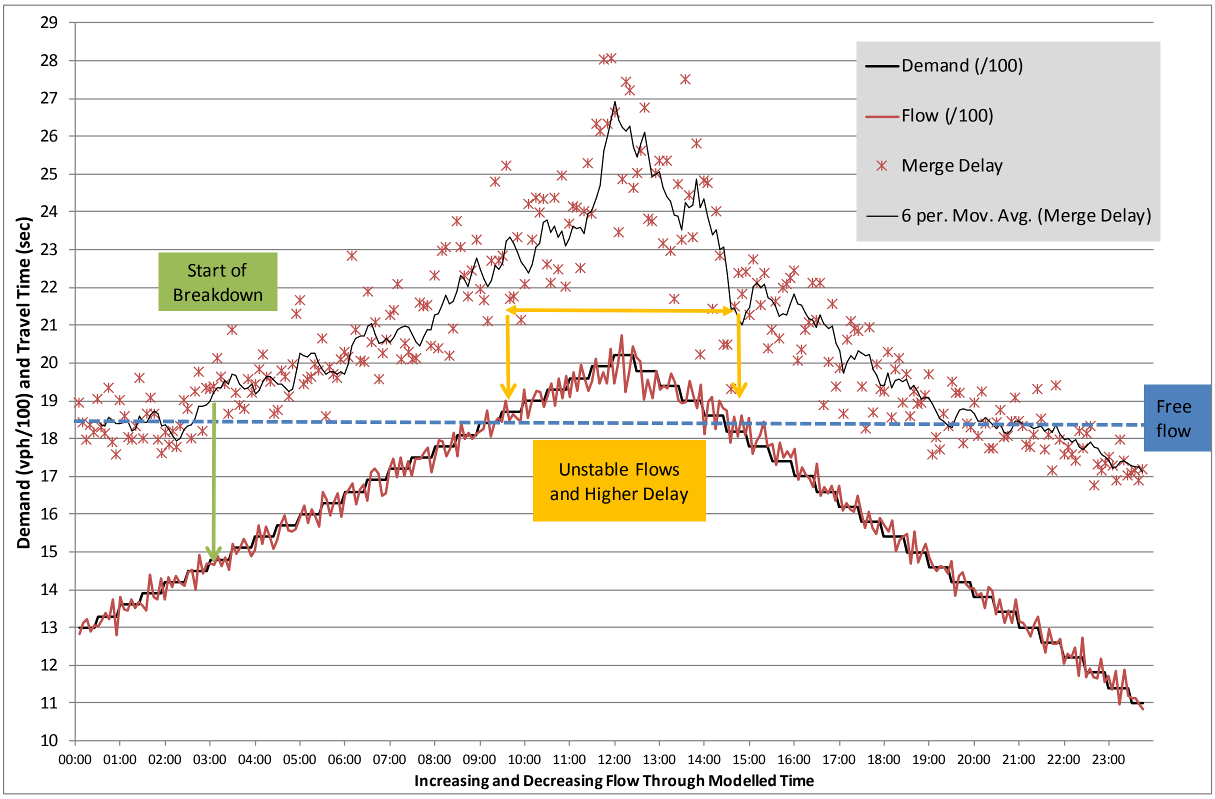
Model performance criteria needed to be identified that would demonstrate and differentiate between the potential benefits and disbenefits for each ITS-assisted merge application, while maintaining commonality for comparison purposes. To ensure that all the key characteristics were adequately assessed the following three performance criteria were chosen:

- The delay (change to travel time) through the immediate merge area at the end of the passing lane measured over 400m (200 upstream and 200m downstream of the centre of the merge). This measure was chosen because one of the key objectives of the ITS-assisted merge application was to reduce delay at the merge.
- The travel time through the length of the passing lane, to reach a point 200m downstream of the merge. This measure was chosen to ensure that the ITS-assisted merge application did not simply re-locate the merge problem to another location within the passing lane.

- Total network travel time through the full length of the model. This measure was chosen as a global measure and to ensure there was no overall significant adverse effect on global journey times.

Figure 7.1 illustrates the operation of an uncontrolled merge. A demand rate of 1,300vph, increasing to 2020vph, and then decreasing back to 1,100vph was used. The demand flows were increased at a rate of 30vph every 30 minutes, and decreased at 20vph every 30 minutes.

Figure 7.1 High-volume 1,200m passing lane – merge delay characteristics



The graph demonstrates the travel time through the merge as the demand flow rises and falls. The graph is fundamental to the assessment of the ITS-assisted merge applications as it captures the key operational characteristics of the uncontrolled merge.

The most important observation is that the merge does not experience a catastrophic failure. During the highest demand flow (2,020vph), the merge still continues to operate, but the travel time during these periods is markedly increased (ie merge delays worsen). This was a significant finding during the project as, from previous research undertaken, there was an expectation that the merge would fail under such high volumes leading to significant queuing and delay within the model. The explanation of this behaviour is that the single lane section upstream of the passing lane acts as a natural bottleneck, which stops the passing lane merge from becoming over-saturated and catastrophically failing during temporary periods of significant flow.

The idea that the merge could still function at these high volumes was supported by a review of the Transport Agency's TMS data and traffic counts for other 2-to-1 merges as follows. The values stated are equivalent hourly flows from max 5 or 15 minute counts:

- Ngauranga intersection to northbound SH1 – maximum of 2,200vph (taken from intersection traffic count 2011)

- Terrace Tunnel, Wellington, southbound SH1 – maximum of 1,964vph (taken from TMS site ref: 01N11074)
- Thermal Explorer Highway, south of Hamilton, southbound SH1 – maximum of 1,884vph (taken from TMS site ref: 01N00559)
- River Road, southbound SH2 – maximum of 2,000vph (taken from TMS site ref: 00200957)
- River Road, northbound SH2 – maximum of 1,976vph (taken from TMS site ref: 00200957)
- Pukerua Bay, northbound SH1 – maximum of 2,028vph (taken from TMS site ref: 01N01045).

There are two other important characteristics to the graph; the 'start of breakdown' and areas of 'unstable flows and higher delay' as follows:

- The 'start of breakdown' has been defined whereby the travel time through the merge begins to significantly increase. This has been identified at a demand flow of approximately 1,450vph.
- The area of 'unstable flow and higher delay' has been defined whereby the effects of such large traffic volumes cause significant increases in travel time. This has been identified as occurring when demand flows are greater than approximately 1,850vph.

The information from the graph has been used to define two conditions that are fundamental to the assessment of the ITS-assisted merge application. First, when traffic flows reach 1,450vph, the ITS-assisted merge application should be in use to manage delays at the merge. Second, when traffic volumes are above 1,850vph, the passing lane should be closed to avoid periods of unstable flow and higher delays. Applying a conservative approach to these values, the operational requirements of the ITS-assisted merge have been defined as:

- the passing lane needs to be closed over 1,750vph
- ITS-assisted merging is required to operate between 1,450vph and 1,750vph.

7.3 ITS-assisted merge concepts

Several ITS-assisted merge concepts have been assessed in order to determine the optimum application. Each of these has been compared on a relative basis as well as against an uncontrolled merge (ie without any merge control). The ITS-assisted merge concepts and a description of their operation are as follows:

- **Dynamic early merge:** Advance warning for vehicles to move to the left rather than continue to utilise the fuller length of the passing lane.
- **Dynamic late merge:** Signing for vehicles to remain in-lane and 'merge like a zip' at the end of the passing lane rather than attempting to move left before the merge.
- **Increase headway:** Investigation into the effect of instruction to drivers to use a longer following spacing.
- **Narrow headway distribution:** Investigation into the effect of reducing the variation in headways used by motorist.

- **Speed reduction:** Active traffic management implementing a variable speed restriction along the passing lane with 100% compliance rate.

Within the scope of this study, each ITS-assisted merge concept has two key challenges: can the operational concept be effectively represented using the microsimulation models? And, in the real world, can driver behaviour be appropriately influenced to comply with the intended operational concept? The answer to the first question, in all cases, is yes – the ability to sufficiently test an idea must be possible for it to be considered. However, the answer to the second question is less certain.

Rather than focus too closely on the likelihood that particular messages will be understood or not, the first round of testing of the ITS-assisted merge concepts assumes a 100% compliance rate. For example, in the dynamic late merge scenario, vehicles in the model are prohibited from changing lanes in the final section of the passing lane and must merge like a zip.

7.3.1 ITS-assisted merge concepts results

Each of the ITS-assisted merge concepts identified above was assessed and measured against a do nothing scenario (ie no ITS-assisted merge application) so the benefits could be objectively assessed. The model parameters were the same too.

Figures 7.2 to 7.4 show the results of each ITS-assisted merge concept when compared with the do nothing scenario. The figures show travel time savings through the merge, passing lane and total network respectively (NB: a higher value is better).

Figure 7.2 Merge percentage travel time savings

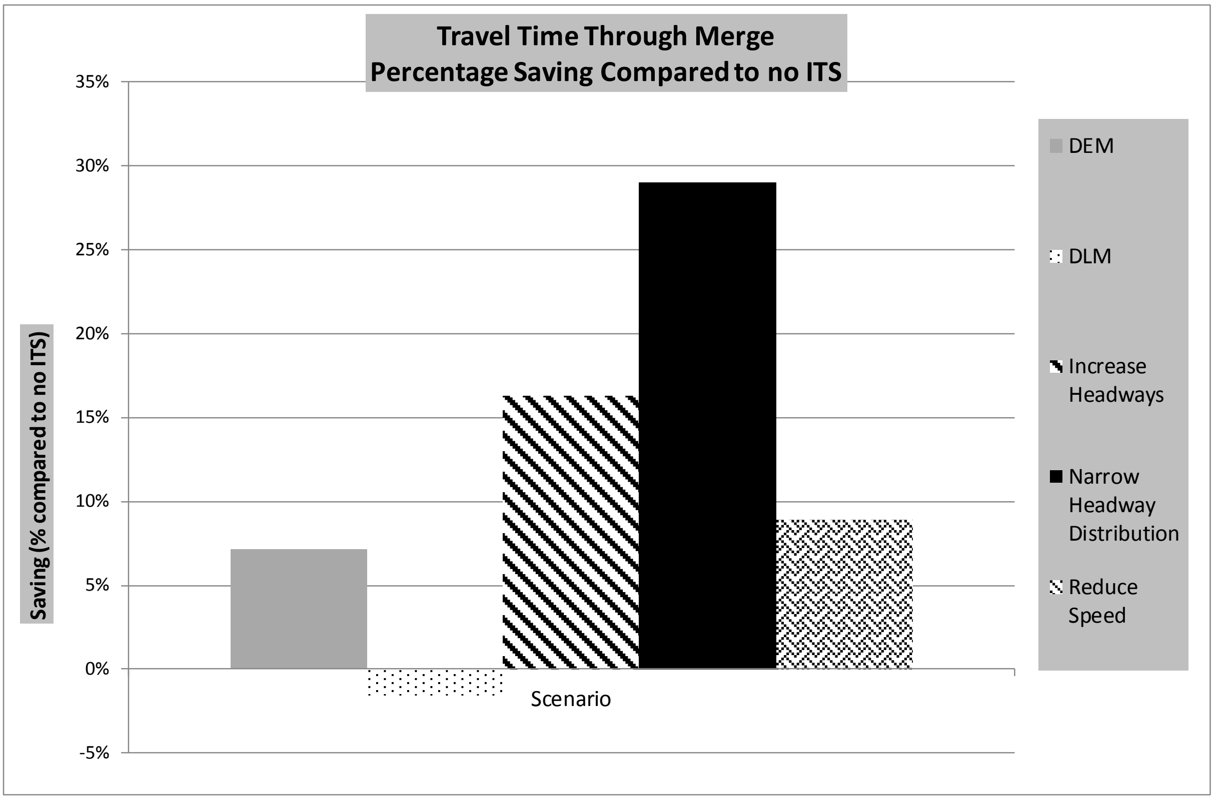


Figure 7.3 Passing lane percentage travel time savings

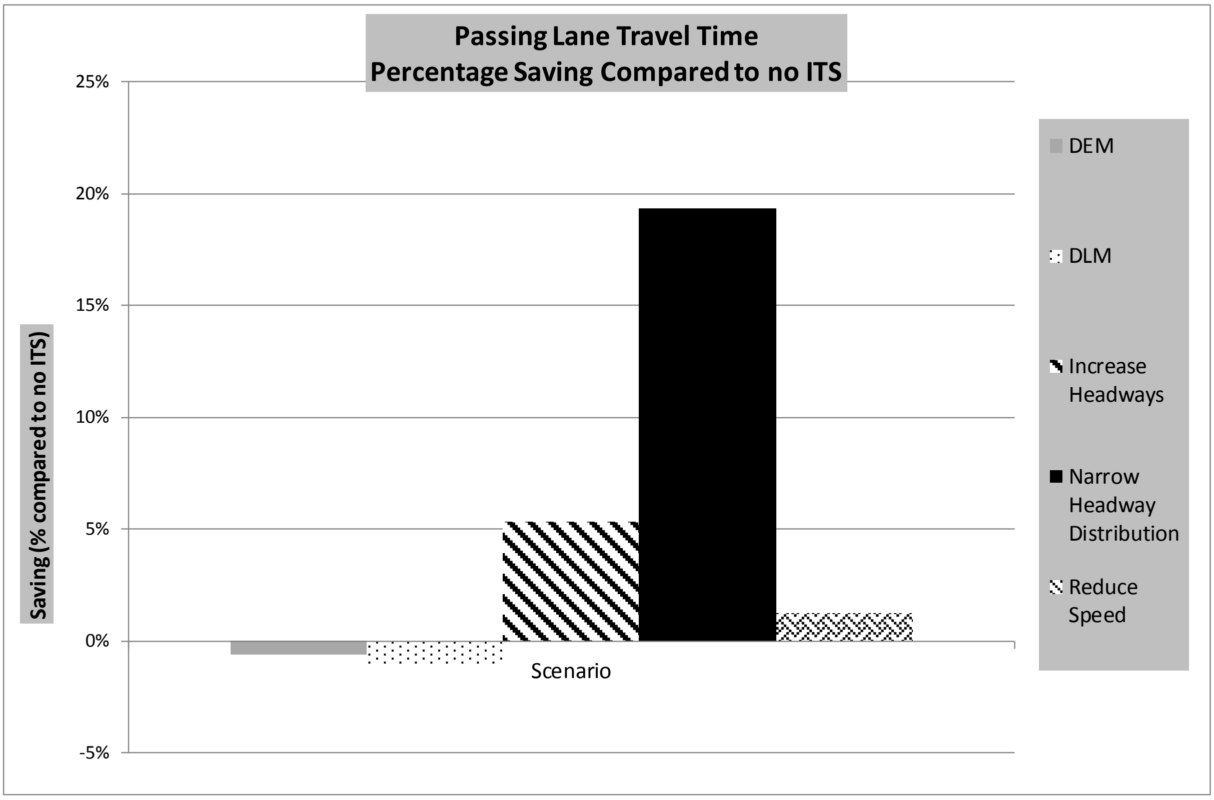
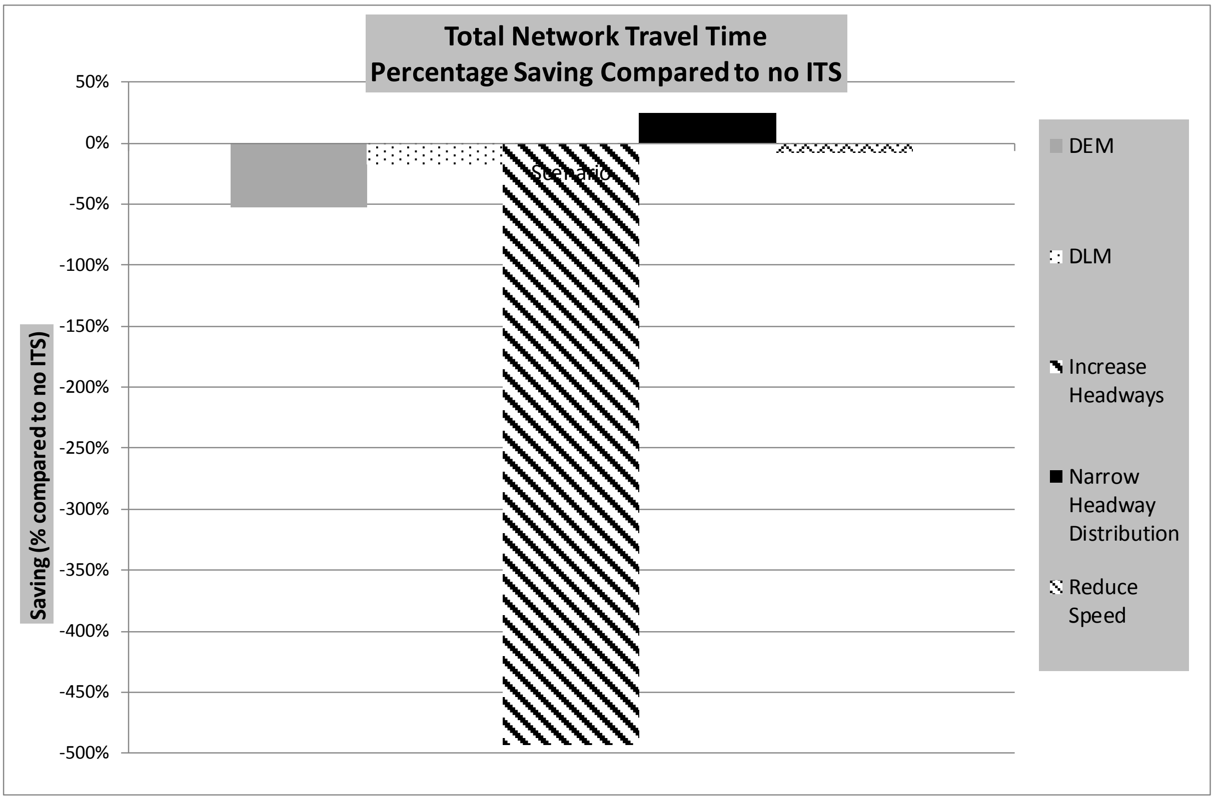


Figure 7.4 Total network percentage travel time savings



7.3.2 ITS-assisted merge results commentary

It is not immediately obvious which solution generated the best outcome on initial assessment of the results. However, taking each concept in turn, the following observations have been made:

- **Dynamic early merge** showed reasonable travel time benefits through the merge (7%) and a minor disbenefit within the passing lane (1%). However, the overall network travel time worsened significantly (52%). The early merge, in essence, shortened the passing lane and restricted the number of overtaking manoeuvres that could occur.
- **Dynamic late merge** showed travel time disbenefits across all the assessment criteria. Travel times were worsened through the merge, within the passing lane and through the total network (2%, 1% and 17% respectively).
- **Increased headways** showed significant improvement through the merge (16%), moderate improvement through the passing lane (5%), but catastrophic failure of total network travel time. The longer headways enabled successful interaction between vehicles (ie greater distances to potential conflict) but reduced vehicle flow rates and increased travel times overall.
- **Narrow headway distributions** showed the most benefits from the modelling with travel times through the merge, passing lane and total network improving significantly (29%, 19% and 25% respectively). There are some techniques, such as 'keep your distance' chevron markings located in the traffic lanes, that can assist in reducing the distribution of headways as described by Greibe (2010) and Tignor et al (1999) – see figure 7.5. However, these, by necessity, must centre the distribution around a higher set point (ie increased headways – see above). It was considered that delivering a narrower headway distribution of sufficient magnitude would not be achieved using ITS technologies, but could still offer potential benefits.

Figure 7.5 Chevron markings in the UK to encourage safer following distances (Glen Koorey)

- **Speed reduction** showed reasonable travel time benefits through the merge (9%), a minor improvement (1%) through the passing lane itself, but a total network travel time disbenefit (8%). While compliance with the posted speed restrictions could not be guaranteed, there was high confidence that motorists would at least know, with certainty, how the infrastructure was informing them to behave.

In terms of identifying the optimal ITS-assisted merge solution, consideration of the measured benefits and the ability to communicate clear and understandable instructions to the motorists was required. On balance, the speed reduction solution was considered to offer the most realistic potential to manage the passing lane operations.

7.4 Preferred ITS-assisted merge application

7.4.1 Description

The initial testing identified that the speed reduction/management system offered the greatest potential. To provide a more realistic evaluation of the operation of the proposed system, a fully dynamic system was developed and linked with the microsimulation traffic model, whereby the speed restrictions were only in force when traffic speeds and flow met particular thresholds (as opposed to the traffic flow input rate of 1,300vph, rising to 2,020vph and falling back down to 1,100vph as was used in the initial assessment). This was simulated using vehicle detection in the model, which measured rising and falling speed and flow thresholds on a second-by-second basis. The speed management system was activated based on smoothed (averaged) rising and falling flow thresholds that were established through iterative testing.

7.4.2 Testing range

To better understand the operation of the proposed solution, minor changes were made to the initial test scenarios as the testing processes evolved and the subtleties of the network operation were better understood. For example, the initial testing included a peak flow (2020vph) level at which the passing lane was likely to be closed. These changes did not affect the overall outcome of the evaluation of the various ITS options.

The passing lane would likely be closed using variable message signs (VMS). One option would be to use a VMS upstream of the passing lane, with a second VMS located halfway down the passing lane to reinforce the message (location dependent on ITS-assisted merge application). This system would require driver compliance. Traditional traffic management methods (cones) could also be used as is the case at a selected number of existing passing lane locations, where high-volume holiday traffic volumes are consistently expected.

To investigate the benefits, the following model characteristics were used to fully establish the operation of the ITS-assisted merge application:

- 1,300m passing lane
- flow range tested from 600vph up to 1,750vph, down to 650vph
- 1,750vph sustained for full simulated hour
- 50vph – 30min demand increment increases and decreases
- ITS solution acting dynamically based on detection of modelled flow rate
- dynamic speed management active above 1,400vph to 1,500vph (precise hourly equivalent flows difficult to establish due to higher levels of platooning).

7.4.3 ITS-assisted merge application results

The key results of the proposed ITS solution are summarised in table 7.1, as compared with the do nothing (no ITS-assisted merge application) scenario.

Table 7.1 Summary of overall performance of proposed ITS solution

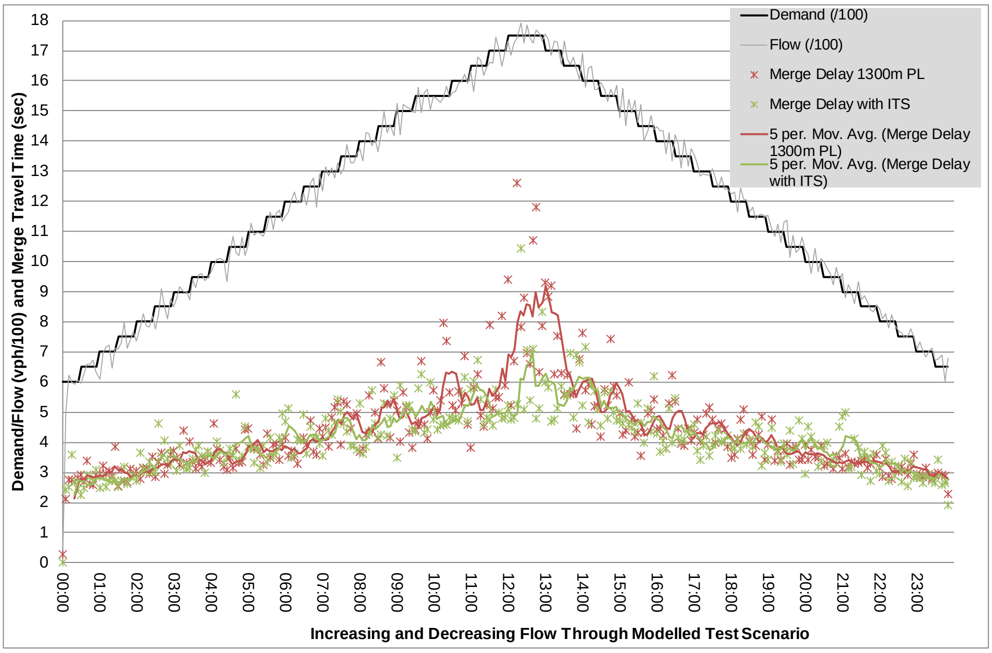
Measurement	Peak range (flows >= 1,460)			Full range (flows >= 650)		
	No ITS	Dynamic speed	Saving	No ITS	Dynamic speed	Saving
Merge travel time	19.2	18.6	4%	17.7	17.5	1%
Passing lane travel time	70.2	70.0	0%	68.3	68.3	0%
Network travel time	621.6	620.8	0%	597.4	597.4	0%

Of note, the disbenefit (8%) in total network travel time measured in the initial ITS testing and identified in figure 7.1 is removed in this refined test (ie there is no net detriment in overall network travel time). This is due to the combination of two effects: the speed management system is not as efficient at higher flow ranges (>1,800vph), and for the initial testing, speed reductions were consistently activated rather than only being activated when conditions were appropriate.

7.4.4 Flow range results

The delay through the merge area (difference between average travel time and optimal 'free-flow' travel time) has been plotted against the flow range (vph divided by 100) for both the ITS scenario, and the do nothing scenario in figure 7.6.

Figure 7.6 Merge delay through flow range



This demonstrates the protection of the higher merge delays offered by the ITS for the key flow range, 1,500vph to 1,750vph.

7.4.5 Summary

The results identified above can be distilled into the following key outcomes:

- Merge travel time:
  - saving during high flows (>1,450vph)
  - saving diminishes when measured across full flow range (ie ITS does not operate below 1,450vph, therefore measuring from 600vph up to 1,750vph and back down presents a dampened return)
- Passing lane travel time:
  - no substantial saving, no net detriment
- Network travel time:
  - no substantial saving, no net detriment.

Whilst the ITS-assisted merge solution delivers benefits, these are small in magnitude and rely on a 100% compliance rate on-road. For these reasons, it is not anticipated that ITS-assisted merge applications should be deployed in New Zealand, within the passing lane or 2+1 environment. The predicted saving will not be achieved if driver compliance rates are similar to those in other electronic variable mandatory speed environments (eg Wellington's urban motorway). The rural environment (lack of enforcement officers) also results in an expected lower compliance rate.

The benefits of managing merges at work sites in the USA are achieved in a significantly different environment from New Zealand passing lanes, often managing vehicles for several kilometres upstream of the lane merges.

New Zealand drivers are comfortable with passing lane operations and generally use them courteously by keeping left unless overtaking. When traffic volumes increase, the utilisation of the passing lanes actually decreases to avoid conflicts at the merge. This 'self-regulation' is not observed by all drivers, but it does represent the behaviours of a significant portion of passing lane users, particularly weekday commuters, which offers a 'natural' merge protection operation.

8 Economic evaluation

In order to establish the economic evaluation of high-volume passing lane route treatments and 2+1 layouts, the magnitude of the associated costs and benefits needs to be determined. In terms of cost, scheme construction and maintenance activities are involved. For the benefits, travel times, frustration, VOCs and safety are all key attributes. These are discussed further in sections 8.1 and 8.3.

8.1 Construction costs

8.1.1 Passing lane construction costs

Average passing lane costs from the EEM (note, all values quoted are 2011 costs) show generalised costs for the purposes of developing a passing lane strategy. These costs are based on a 1km passing lane. The EEM notes there is significant cost variation in passing lanes, with costs varying from $170/m[3] to $1100/m generally[4], but can be up $2,400/m. As such, it is recommended that passing lane costs are determined on a case-by-case basis. Factors that may have a significant impact on cost are: soft ground, intersection improvements, significant earthworks, service relocations, large culvert or drain extensions. The average costs specified in table A7.4 of the EEM are reproduced in table 8.1 (updated to 2011 costs):

Table 8.1 Passing lane construction costs (2011)

Terrain type	Unit cost (/m)
Flat	$350
Rolling	$440
Hilly	$700
Mountainous	$1,100

8.1.2 2+1 roadway construction costs

In many ways, 2+1 road costs are very similar (on a per-length basis) to those found for passing lanes. The key differences will be whether a median barrier (and its associated widening) is used and whether grade-separated intersections are provided.

Potts and Harwood (2003) reported that 2+1 roads in Finland (without median barriers) had construction costs on average about 10% higher than for ordinary two-lane roads. For 2+1 roadways with median barriers, the construction costs were about 15% to 30% higher. Bergh and Carlsson (2000) quoted a cost for replacement of a ‘wide’ two-lane road with a 2+1 WRB road as 2 million Swedish Krona/km (~NZ$500/m in 2012).

[3] Costs are given in NZ$ unless otherwise indicated.

[4] Costs updated from 2005 to 2011.

8.1.3 Wire rope barrier and associated widening construction costs

Australian research undertaken by McTiernan et al (2009) of international experiences in reported costs associated with WRBs (Austroads 2009) equate to an estimated $200/m for installation[5], with no allowance for any necessary widening of the roadway.

Several median WRB projects have been used to give an indication of costs, although these projects involve median and shoulder widening, pavement reconstruction and other safety improvements in addition to the WRB. The two projects considered are:

- SH1 – Longswamp to Rangiriri
- SH2 – Silverstream to Moonshine.

Based on the project description and overall costs, an estimation of the component cost specific to the WRB (ie does not account for median widening) has been made, as summarised in table 8.2.

Table 8.2 Estimated WRB installation costs from state highway projects (2011)

Project	Length road	Total unit cost (/m)	Estimated % WRB component	Estimated WRB unit cost (/m)
SH1 – Longswamp to Rangiriri	9km	$980	30%	$294
SH2 – Silverstream to Moonshine	3.4km	$741	30%	$222

These costs are relatively consistent with the $200/m determined from Austroads, based on overseas experience. Considering these Austroads findings and the previous New Zealand experiences of WRB installation, a cost of $250/m (2011 value) has been assumed for the installation of the WRB.

The 2+1 roadway design shown in figure 4.1 identifies a 2m median necessary to house the WRB with some accommodation for barrier deflection. Building on the widening costs associated with passing lanes and 2+1 roadways, it has been estimated that cost of $250/m would be adequate for this 2m carriageway widening component. This brings the total cost for median WRB to $500/m (2011 value).

8.1.4 Maintenance costs

Passing lanes and 2+1 roadways introduce additional traffic lanes, and hence sealed road surface, to the road network. As a result, it is expected that the ongoing maintenance costs of resealing, pothole repairs, etc will increase along these sections. The additional signage and road markings at the start and end of passing lanes will also require additional maintenance costs for their upkeep. It has been estimated that maintenance for passing lane pavements (including 2+1 layouts in a single direction of travel) is 1% of the passing lane construction cost.

It is likely that WRBs would increase other maintenance costs; as reported earlier, additional repairs to barriers through vehicle strikes is one particular cost to consider. Bergh et al (2005) estimated that typical WRB repair costs in Sweden amounted to ~50k to 70k Swedish krona per km per year (~$11,000 to $15,000/km in 2012).

[5] Costs updated from 2002 to 2011, and factored for New Zealand currency

McTiernan et al (2009) reviewed international experiences and estimated between $28 to $60/m (2009 values) for WRB repairs as a result of a vehicle strike[6]. To determine the annual maintenance cost, this value needs to be factored to take account of the frequency of WRB collisions.

The collision frequency used in Australian research is similar to that observed in New Zealand, being approximately 0.7 collisions per 10^6km travelled. Having considered the Austroads international findings and New Zealand's observed collision rate, a cost of $10/m per year has been assumed for maintenance of the WRBs. The maintenance costs are summarised in table 8.3.

Table 8.3 Maintenance cost summary

Component	Maintenance cost estimate (2011)
2+1 roadways and passing lanes	1% of initial construction cost per annum
Wire rope barriers	$10/m per annum

With standard (8%) discounting (Transport Agency discount rate at the time this research was completed) over the lifetime of the project, the total present-value maintenance costs would typically be equivalent to ~11 times these values, eg WRB maintenance would cost ~$110/m (in present-day 2011 dollars) over the lifetime of the installation.

It should be noted that construction and maintenance delays can influence the success of road widening projects and these should be estimated as part of the detailed project evaluation process.

8.2 EEM option evaluation process

For the purposes of the option evaluation process (see appendix A), a 2011 value of $1,000/m for passing lane construction and maintenance has been assumed. In the hypothetical example of a 2+1 roadway without a WRB (ie a similar arrangement to passing lane treatments), this same cost is considered to be applicable. The option evaluation process developed for this study features an adjustment equation whereby a construction/maintenance cost estimate can be inserted into the process to 'overwrite' this $1,000/m value so the corresponding BCR can be determined based on a more refined cost. This is required for any passing lane or 2+1 roadway as the actual construction costs for any scheme will vary significantly depending on the gradients, ground conditions and associated works involved. For a 2+1 roadway with a WRB (in line with the proposed design), there will be a further $250/m for the additional widening necessary for the WRB, and a further $350/m required for the WRB construction and maintenance alone. These costs in 2011 values are summarised in table 8.4.

Table 8.4 Construction/maintenance cost summary

Component	Construction and maintenance cost ($/m)	Comments
Passing lane	$1,000	1% maintenance cost ignored
2+1 roadway (with WRB)	$1,600	Includes necessary median widening
WRB only	$360	
2+1 roadway (without WRB)	$1,000	Not recommended

[6] Equivalent cost in NZ$ based on AU$1 = NZ$1.2

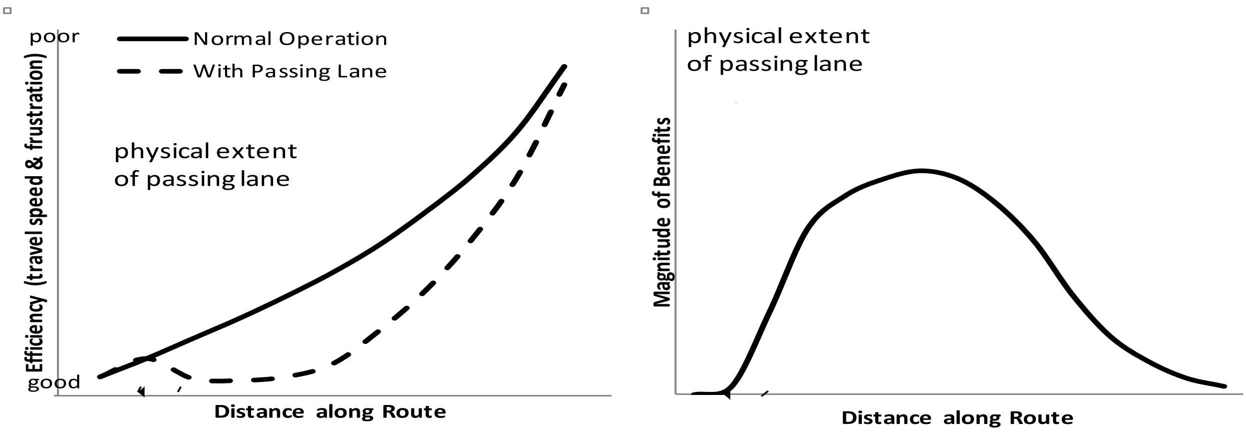
8.3 Measurement of benefits

In the EEM, the economic benefits of passing lanes are obtained in four ways:

- Reductions in travel time due to faster journeys after overtaking slower vehicles.
- Reductions in crashes (especially overtaking, head-on, rear-end) due to safer overtaking opportunities (and prevention of head-on crashes if median barriers are present).
- Reductions in driver frustration from less time spent following slower vehicles.
- Changes in VOCs due to faster journey speeds (typically a slight disbenefit, but with a small possibility of positive benefits if passing at a significantly smooth journey speed compared with the previous situation).

These benefits typically apply only in the direction of travel (although crashes may affect traffic in both directions) and can apply both within passing lane sections and for a considerable distance downstream. Figure 8.1 illustrates how the passing lane efficiency benefits may accrue along a route.

Figure 8.1 Profile of passing lane efficiency savings



With a regularly repeating series of passing lanes (including 2+1 configurations), a new set of benefits will begin each time the next passing lane starts. In many cases this new passing lane will begin before the downstream benefits of a single passing lane have finished accruing.

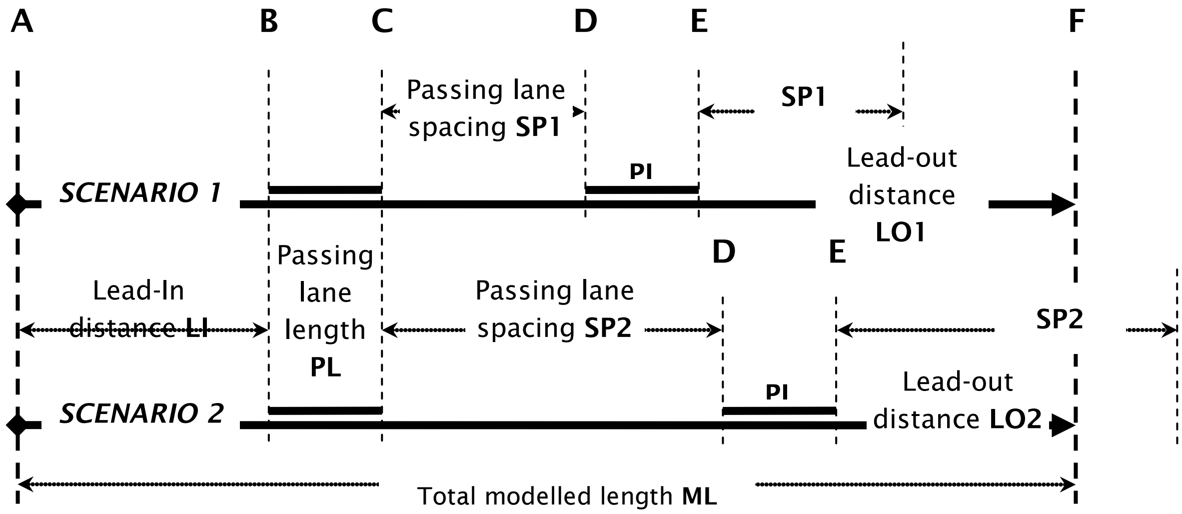
Sections 8.3.1 to 8.3.4 describe the principles and considerations undertaken to establish how the various benefits apply to the differing passing lane scenarios.

8.3.1 Conceptual model for passing lane benefits

To analyse the effects of different passing lane configurations, a series of models were run with different traffic volumes, terrains and percentages of heavy vehicles, as described in chapter 1. Other than the do minimum base cases (no passing lanes) each option included two passing lanes of the same specific length, and a specific spacing between passing lanes. For consistency of comparisons between outputs, all models at the same AADT were run over the same total modelled length of road and with the same lead-in length prior to the first passing lane. This approach meant that the lead-out distance downstream of the second passing lane varied between models of different passing lane lengths and spacings. Figure 8.2

illustrates two typical scenarios as represented in the various models. In this case, the same passing lane length has been used, but two different spacings, SP1 and SP2, are being tested. The points A to F illustrate the main transition points within the models, as described previously in figure 6.3.

Figure 8.2 Typical modelled passing lane scenarios



In the case of scenario 1, the lead-out distance is longer than the passing lane spacing; therefore in reality another passing lane might have been installed as part of a longer series of passing lanes, and a new set of benefits accrued.

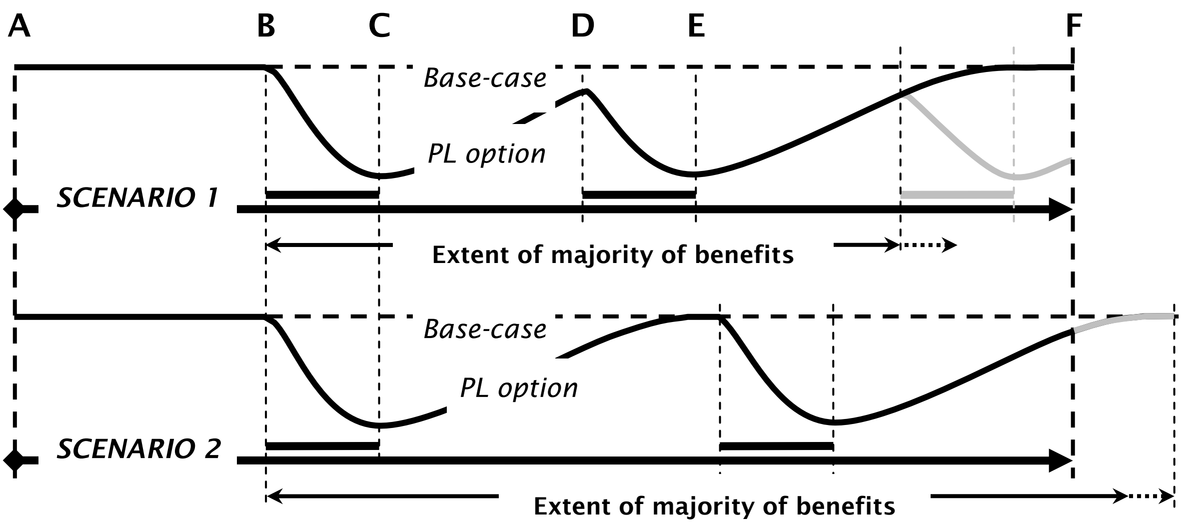
In the case of scenario 2, the lead-out distance is less than the spacing to the next passing lane. Therefore there might be some additional benefits from the second passing lane not accounted for in the model.

Ideally the lead-out length would be equal to the spacing. Setting up the models in this fashion is not practical as it would require every do minimum network to vary in length and results between length/spacing option scenarios would not be comparable. The benefits calculated for each modelled length need to be marginally scaled up or down to ensure a consistent set of benefits per km for comparison across all cases.

8.3.2 Travel-time benefit scaling

Figure 8.3 (not to scale) illustrates the hypothetical effects of the two passing lane option scenarios on the measurement of efficiency savings (travel time and following), compared with the do-minimum base case with no passing lanes. In scenario 1, the likely benefits of a third (and any subsequent) passing lane within the modelled length (as shown in grey) will not be captured by the model. In scenario 2, the diminishing benefits of the second passing lane further downstream beyond the end of the modelled length (again shown in grey) will not be reflected in the modelled length.

Figure 8.3 Typical effects of passing lanes on following %



This issue was investigated in some detail and several approaches tested to account for the differences in benefit accruement in the lead-out section. Initially, the theory was validated by running an AADT 10,000 model with an additional long downstream length. This demonstrated that travel time savings accrue a considerable distance downstream of the passing lane at a diminishing rate.

The most reasonable method for assessing this phenomenon assessed the *relative* difference in travel time benefits between options with the same AADT and passing lane length, but with different spacings. Starting with the base case of the passing lane configuration with the largest spacing, invariably any option with smaller spacings will generate greater travel time savings. This however has to be balanced against the extra cost per km of having more frequent passing lanes. The process is as follows:

- For each passing lane option calculate the 'core' travel time saving over the length of the model, relative to the do minimum.
- Calculate the extra savings from more passing lanes at shorter spacings, relative to the maximum spacing option: extra travel time = travel time from maximum spacing option – travel time from shorter spacing option.
- Scale the extra travel time by the number of passing lanes possible in the model length: additional travel time saving = extra TT × (ML-LI)/2(PL+SP).
- Calculate travel time saving per vehicle = core travel time + additional travel time.

This approach provided a logical set of travel time savings and subsequent BCRs.

8.3.3 Crash cost saving benefits

In an economic evaluation of passing lanes in New Zealand, the typical crash rate within each passing lane is assumed in the EEM to be 75% of the normal two-lane crash rate, ie a 25% crash reduction. This reduction then declines linearly downstream of the passing lane until the normal two-lane crash rate is reached again. Typically it is assumed that this linear decline will occur over 5km to 10km beyond the passing lane.

In the same way as described above for travel time savings, crash benefits are under-estimated due to the likely benefits due of not incorporating the savings from any third (or subsequent) passing lane over the fixed analysis length.

The calculations for crash savings are easier to determine based on the effective downstream length, EL. A determination of the average crash rate can be calculated by means of a weighted average of up to three road sections:

- the passing lane length, where it is usually assumed that a constant 25% reduction in the normal two-lane crash rate will apply
- the downstream length where the 25% reduction is linearly reduced over the length EL (this may be cut-off if the passing lane spacing SP is less than EL)
- the remaining downstream length (SP-EL) until the next passing lane starts, where the normal two-lane crash rate applies (this section only applies if EL<SP).

The 25% crash reduction takes no account of the length of the passing lane. It is entirely reasonable that a shorter passing lane that does not provide sufficient length for many desired overtakings will not generate the same level of safety improvements. The literature is surprisingly bereft of research on the subject of passing lane length and safety. Although there are plenty of studies looking at the operational effects of having short or long passing lanes, there does not appear to be anything showing a direct link to the safety implications.

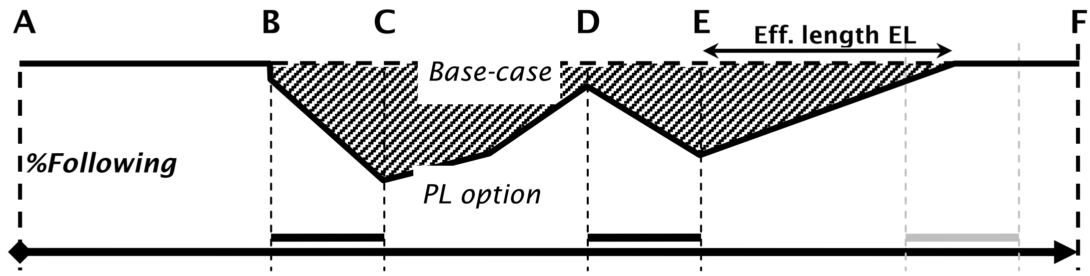
The operational studies and testing completed in this research, indicate that passing lanes of 1000m to 1200m provide optimal passing opportunity (with basic consideration of passing lane cost). It is logical to assume that a similar length is required to get the maximum 25% crash benefit, otherwise this value could be proportionately reduced in relation to the actual passing lane length supplied.

8.3.4 Driver frustration benefits

Driver frustration benefits are derived from the ‘time spent following’ information. Research by Koorey et al (1999) established a ‘willingness-to-pay value’ for the provision of passing lanes of 3.5 cents (1999 value) per vehicle per kilometre of constructed passing lane (this is in addition to other benefits such as travel time savings). This benefit is applied to all vehicles that are freed from a platoon at the passing lane over the length they remain free from a platoon. The value of 3.5 cents/veh/km should only apply to vehicles travelling in the direction of the passing site. The vehicle-km to apply the willingness-to-pay factor to, should be determined by multiplying the traffic volume by the analysis length and the change in time spent following.

The area difference in %following between the base case and each passing lane option (as illustrated by the shaded area in figure 8.4) reflects the vehicle-km to which the frustration factor should apply. The %following data at various locations can be obtained from the modelled outputs through the placement of detectors which measure vehicle headway (following time in seconds) at set distances through the route. Again, depending on the type of scenario, each case may need to be scaled up or down to account for under/over-estimation of the average long-term reduction in %following.

Figure 8.4 Typical effects of passing lanes on driver frustration

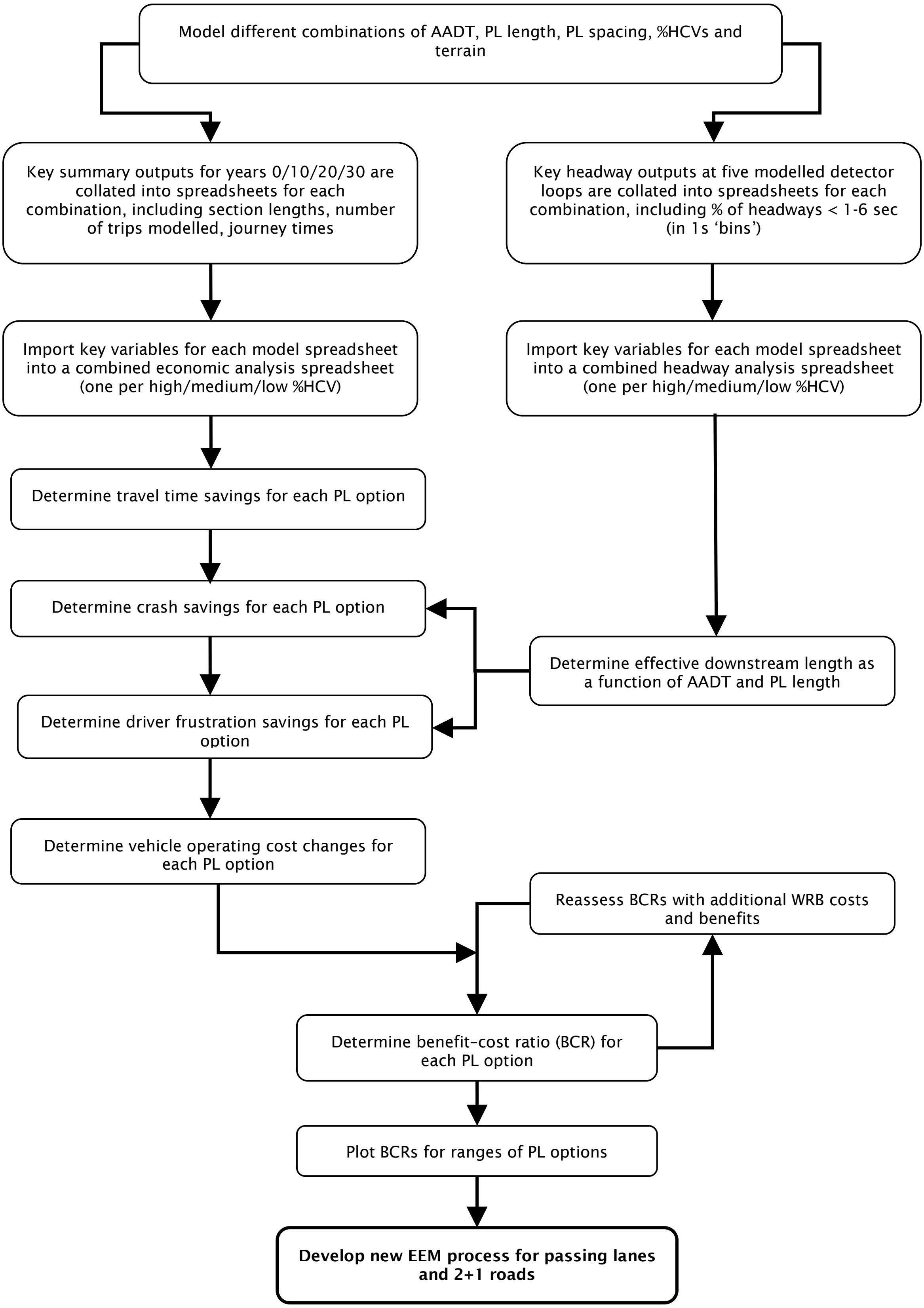


8.4 Analysis process for economic evaluation

Based on the above investigation and theory regarding costs and benefits of passing lanes and 2+1 roadways, a methodology was developed to derive typical BCRs for different configurations of the key input parameters.

Figure 8.5 summarises the general process used for the analysis. More details about some of the main steps are described in the following sections. It should be noted that this analysis was undertaken (during the research) with the then appropriate 2011 EEM values. The EEM has since been updated with new factors and new costs (2012 values).

Figure 8.5 Methodology for deriving new economic evaluation processes



8.4.1 Travel time savings for each option

The total average journey time in each direction for the entire model length has been collated for each model run. These are compared with the do-minimum option for the given AADT band to determine the basic travel time savings. As described in section 8.3.2, these savings are then adjusted to produce relative travel time savings per vehicle per km. The savings are then scaled up by half the AADT (in the direction of travel) and multiplied by the standard EEM hourly rural travel time cost ($23.25, 2011 value) and by 365 days to produce an annual total of travel time savings.

8.4.2 Crash savings for each option

The average crash rates for the passing lanes and their downstream sections were derived, as described in section 8.3.3. For the purposes of the benefit–cost factoring as described in section 8.3.3, a minimum passing lane length of 1,200m was used to assign full (25%) crash reduction. Passing lanes shorter than this had their crash reduction rate reduced in proportion to their length, eg a 600m passing lane had only a 12.5% reduction applied.

8.4.3 Driver frustration savings for each option

Driver frustration savings are based on the change in %following (between do minimum and option) throughout the model (as recorded in the headway data). The area under the graph between each of the detectors is used to determine the average change in %following. Downstream of the last detector (end PL2) it is assumed that the %following difference goes back to zero over the calculated downstream distance.

8.4.4 Vehicle operating cost changes for each option

VOCs were calculated from the EEM VOC costs per km for rural roads from the modelled average travel speeds. Typically this produces a minor disbenefit (difference between do minimum and option) due to higher speeds in the with passing lane scenario. The exception to this is where the do minimum has significantly fluctuating journey speeds and the option smooths out speeds.

8.4.5 Annualisation and discounting

The process for the economic components described above was repeated for years 0, 10, 20 and 30. The average arithmetic growth in the economic outputs over the 30 years was calculated (for passing lane evaluations, typically this does not match traffic volume growth rates) and suitable discounting growth factors determined to calculate the present value of total travel time benefits over the 30-year analysis period.

8.4.6 Determine benefit–cost ratio of each option

All benefits and costs over the 30-year evaluation period were summed to calculate the final BCR for each passing lane length and passing lane spacing scenario tested.

8.4.7 2+1 Layouts and treatment of WRBs

WRBs are recommended on 2+1 layouts (see section 4.1) as they provide additional crash saving benefits (see section 3.4). BCR analysis has been completed considering the additional crash savings balanced with the additional construction and maintenance costs.

The outcome from the analysis indicated that there is a general increase in BCR of between 0.1 and 0.2 for the use of WRBs with 2+1 roadways. It is important to note that the use or otherwise of WRBs results in no significant change to the optimal length/spacing outcomes.

9 Operational results

9.1 Overview

Throughout this research project nearly 2,000 individual traffic models were run to establish the benefits of high-volume passing lanes and 2+1 roadways. Each model had a combination of characteristics (variables) and a resultant set of benefits. These are identified in table 9.1 below.

Table 9.1 Traffic model characteristics/variables and benefits

Model characteristic/variable	Model benefit
Terrain gradient	Travel time
AADT	Vehicle operating cost (typically negative benefit)
HCV component	Driver frustration
Traffic growth year	Safety
Lead in length	
Passing lane length	
Passing lane spacing	
Run out length	

In its simplest form, the benefits of a particular passing lane arrangement will always be greatest when the passing lanes are longer and spacings are shorter, as this gives the most opportunities for faster vehicles to overtake slower vehicles and remain free from platoons. However, longer passing lanes with shorter spacings will also result in the greatest implementation costs. Reporting the passing lane benefits without consideration of the passing lane costs is meaningless and potentially misleading.

A subtlety of this analysis is that the benefits of a given passing lane length can only be assessed with an associated and defined downstream spacing. Consider the fictitious situation where two 1km passing lanes are separated by a 500m single-lane section. All the faster vehicles will have performed their overtaking manoeuvres in the first passing lane (resulting in the first passing lane delivering high benefits) meaning that the second passing lane is unlikely to be used (resulting in the second passing lane delivering low benefits).

For each traffic model, the research considered the magnitude of the benefits, the magnitude of the costs and developed a set of results which identified the optimum passing lane length-and-spacing-combination for a given terrain, AADT range, HCV component and traffic growth forecast. To account for the subtleties noted above, a wide range of combined length and spacing tests were run as separate models and the BCRs of each scenario calculated as described in the sections discussing the measurements of benefits and economic evaluation analysis procedure (sections 8.3 and 8.4).

A transport practitioner looking to implement a passing lane or 2+1 strategy for a given section of highway can identify the specific terrain, AADT and HCV composition to establish the viable range and optimal economic return from length and spacing combinations. Further details about this process, including a step-by-step guide for implementing the results of this research, are contained in appendix A.

The results are provided in section 9.2 as a series of contour plots for a given terrain, AADT range and HCV composition. The contour plots identify BCR ranges for the given passing lane length and spacing combinations, with the optimum combination clearly identified. The magnitude of the BCR (ie the specific values shown) is based on the construction cost of the passing lane or 2+1 treatment (in one direction only) of $1,000/m (2011 values). If actual construction costs differ from this value (as would be expected to be the case in the majority of instances), the scale of the BCR will change accordingly, but, importantly, the optimal length and spacing combination will remain unchanged.

9.2 Contour plot results

A sample of the length/spacing contour charts has been provided in figures 9.1 to 9.4 to demonstrate the principle of the EEM method and the outcomes of the modelling and economic evaluation. The comprehensive set of contour plots (24 separate plots) is contained in appendix A.

The contour plots show passing lane spacing on the x-axis and passing lane length on the y-axis. The corresponding BCR for each passing lane length and spacing combination is identified by colour/shade.

Figure 9.1 Flat/rolling terrain, AADT 10,000, medium heavy vehicle percentage

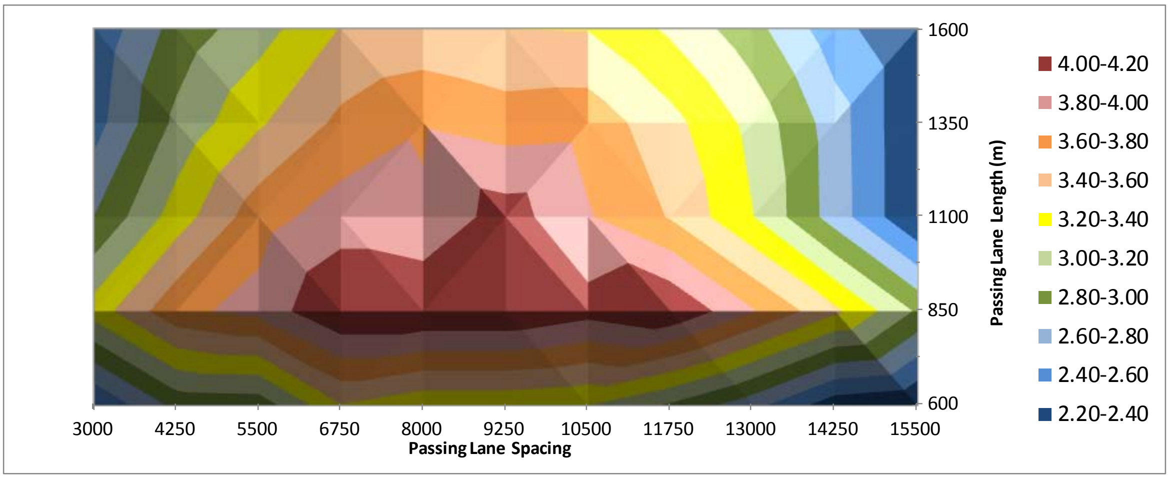


Figure 9.2 Flat/rolling terrain, AADT 15,000, medium heavy vehicle percentage

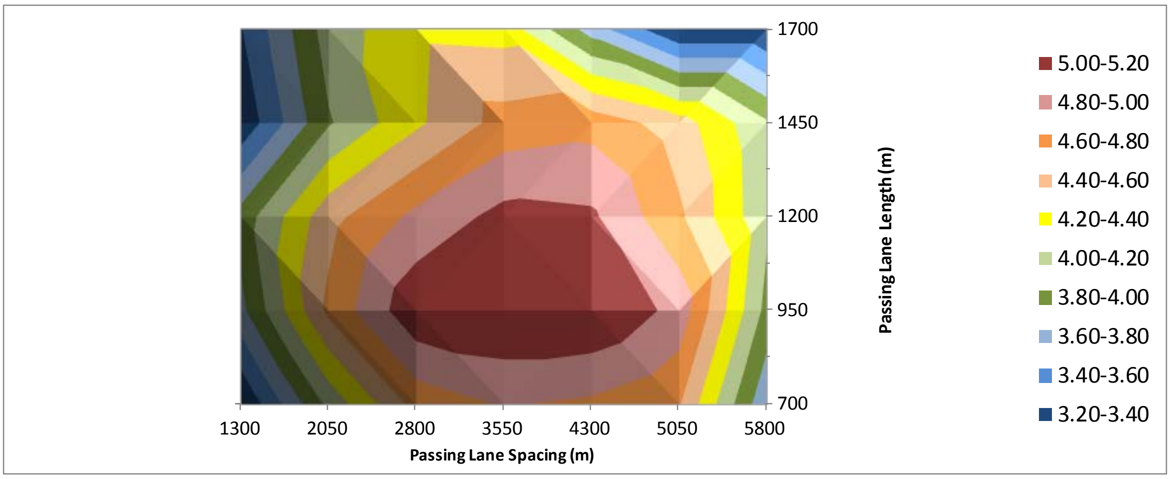


Figure 9.3 Flat/rolling terrain, AADT 20,000, medium heavy vehicle percentage

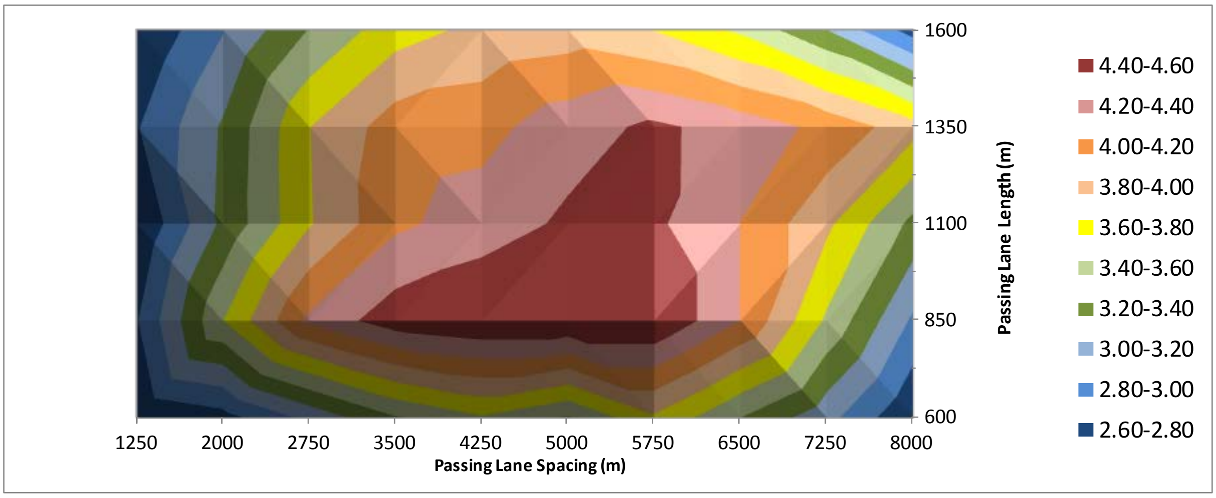
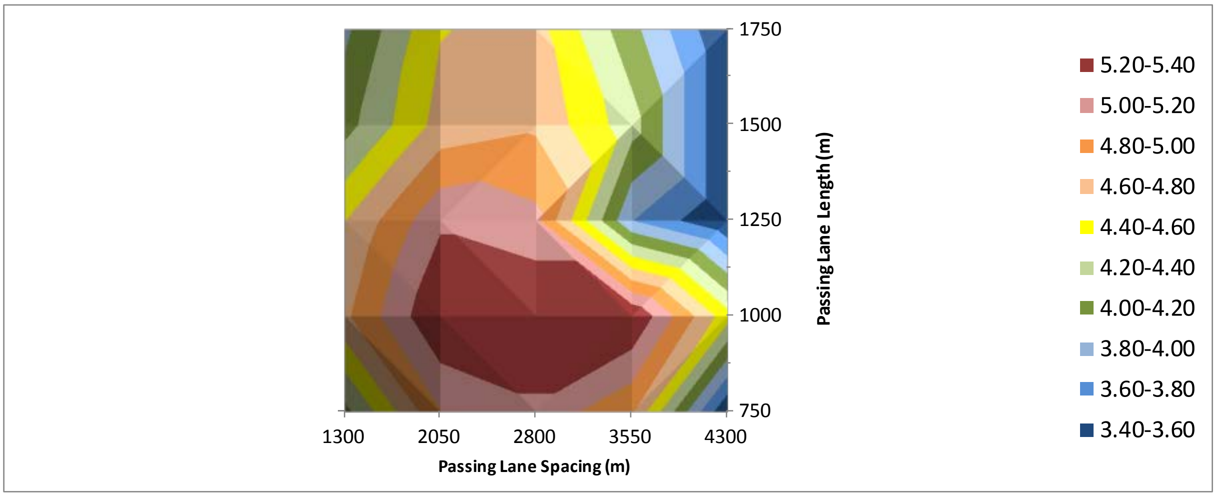


Figure 9.4 Flat/rolling terrain, AADT 25,000, medium heavy vehicle percentage



10 Conclusions and recommendations

10.1 Conclusion

This research project undertook a comprehensive literature review in relation to New Zealand passing lanes, international 2+1 roadways and a range of related subject matters. Through consideration of the findings, an assessment and design activity was undertaken to propose a 'New Zealand 2+1 roadway design standard' incorporating international best practice combined with the experience of existing 'tried and tested' New Zealand standards. The proposed design and international findings were subsequently interrogated to establish the likely safety characteristics for 2+1 roadways in the New Zealand context, which fully addressed the second research objective.

A comprehensive data collection process was undertaken at three existing passing lane locations which featured the specific traffic characteristics and network topographies necessary to obtain a range of vehicle/driver behaviours. Traffic models representing the survey sites were subsequently developed, with core, common parameters identified for use in a 'generic' model which was used to model the range of variables so their outputs could be processed into economic benefits.

Extensive microsimulation traffic modelling (S-Paramics) was undertaken to predict the operational benefits (travel time, crash cost savings, VOCs (typically a negative benefit), driver frustration for a range of passing lane length and spacing combinations covering a range of terrains, AADTs, HCV compositions and traffic growth factors. This was achieved using nearly 2,000 individual traffic models including a range of do nothing options against which the benefits of individual scenarios were directly compared. This process addressed the first research objective.

A separate traffic model was used to assess a range of ITS-assisted merge concepts which sought to manage merge conflicts to enable higher merge flow rates during peak operating periods. The findings from this exercise closed out the third research objective and confirmed that, particularly given a change in driver behaviour when traffic volumes approached their maximum peak demand, merges could operate at reasonably high traffic flow rates. The fact that the flow in the passing lane is restricted by the flow in single-lane section upstream of the passing lane meant that catastrophic merge failure ceased to occur under 'normal situations' (ie when free from crashes). The theoretical benefits from the optimal ITS-assisted merge application resulted in a low (4%) travel time saving through the merge (only). These results only hold if 100% of motorists comply. Given this unlikely situation, and the rural environment in which these will operate, the actual operational benefits will be somewhat lower than theoretical benefits.

The development of passing lane and 2+1 construction costs, and translation of traffic model outputs into economic benefits enabled the optimal length and spacing combinations to be established for a given terrain, AADT flow range and HCV composition. These findings provide useful guidance for transport practitioners and are developed and explained in detail in appendix A, satisfying the fourth research objective.

10.2 Recommendations

The three main outcomes from the research are associated with determining the New Zealand design and safety characteristics of 2+1 roadways, the ability to manage merge traffic and the identification of optimal passing lane length and spacing combinations for a range of given input parameters.

Although not requiring the use of ITS-assisted merging, the scenario testing identified potential merge benefits in reducing the distribution of headways within the passing lane. It is unlikely to be possible to convey this message to motorists as they make use of the passing lane/2+1 roadways using interactive technology systems. However, techniques such as 'keep your distance' chevron markings located in the traffic lanes, can assist in reducing the distribution of headways as described by Greibe (2010) and Tignor et al (1999). It is recommended that a similar arrangement is trialled at an existing, high-volume passing lane to determine if similar benefits can be achieved in the New Zealand environment. The low cost of such an application (white markings and a static sign) is particularly supportive of this recommendation.

Merge behaviour could also be used to determine surrogate measures for safety; for example Yang and Ozbay (2011) investigated conflicts at freeway on-ramps to estimate rear-end crash rates. Simulation models, such as the Surrogate Safety Assessment Model (Gettman et al 2008) have been developed to attempt to relate simulated conflicts with observed crash rates, and this might be a promising area of further investigation for 2+1 road safety design.

The identification of optimal passing lane length and spacing combinations (as described in appendix A) is significantly more advanced than previous economic assessments of passing lanes strategies. It is recommended that the process be adopted within the EEM and duly replace/complement appendix A7 of the existing EEM (noting that since the research was undertaken the EEM has been reviewed and updated – August 2013).

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Appendix A: Option evaluation process

A1 Introduction

This process is designed for practitioners to develop an economic evaluation (benefit–cost ratio (BCR) for:

- 2+1 layouts[7] on flat/rolling road gradients
- high-volume passing lanes in series on mountainous road gradients
- high-volume isolated passing lanes on flat/rolling road gradients within short 5km to 10km state highway sections.

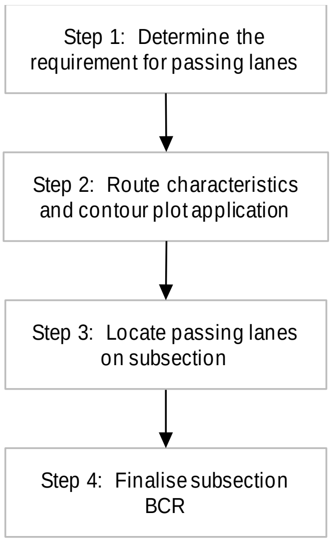
The process is applied in one direction at a time to two-lane rural, 100km/h, state highway environments for annual average daily traffic (AADT) between 5,000 and 25,000 vehicles per day (two-way).

This process is based on placing passing lanes in one direction of travel. If passing lanes are required in both directions (likely for most instances) the process should be repeated for the other direction of travel. The scheme BCR would be a mean average of the BCRs for both directions of travel.

The process is not valid for routes in an urban environment, four-lane sections, and where AADT volumes are outside the range specified above. If passing lanes are a preferred strategy for a route outside this range, it is likely a site-specific investigation will be required rather than application of this process. Further details are provided in section A6.

This process provides considerations, advice and guidance associated with the location of passing lanes and uses a four-step process outlined in figure A.1.

Figure A.1 Overview of process



[7] '2+1 layouts' should be considered as consecutive, alternating passing lanes

The process is based on detailed microsimulation traffic modelling (calibrated and validated based on three representative sites) used to establish the relationship between economic efficiency, passing lane length, and spacing between adjacent passing lanes for a given AADT volume, terrain, traffic growth rate, and percentage of heavy vehicles. The economic efficiency of passing lane schemes is measured in terms of:

- travel time savings – due to faster journeys after vehicles overtake slower traffic
- reduced driver frustration – due to dispersing platoons and less time spent following
- VOCs – changes to fuel use, car maintenance etc due to faster and/or smoother speeds
- crash savings – reductions in crashes due to safer overtaking opportunities.

Travel time savings incorporate the majority of efficiency benefits of passing lane schemes. These savings accrue from the start of the passing lane and continue for some distance downstream of the facility. There are also crash savings as identified in section 8.3.3. The process has been developed from detailed analysis which considers the efficiency savings (reductions in travel time and driver frustration) and crash savings of varying passing lane lengths in combination with the optimal location for subsequent passing lanes (spacing).

A1.1 Terminology

The following terminology has been adopted in this process:

- state highway route: rural section of state highway outside of major urban centres
- subsection: section of state highway defined by adjacent major network features over which the procedure is applied
- major network feature: characteristics on a route which influence the definition of the subsection (ie they form subsection boundary), eg gradient, heavy vehicle percentage (%HV), AADT, vehicle behaviour
- minor network feature: characteristics on a route which affects the ability to physically locate a passing lane, eg bridges, forward visibility, minor intersections
- passing lane: a single passing lane facility on a rural state highway in one direction of travel
- passing lane length: distance in metres of a single passing lane facility, in one direction of travel. Measured from the start of the two-lane dashed line marking to the end of the two-lane line marking (assumes merge and diverge design as detailed within the *Manual of traffic signs and markings* (MoTSaM) (NZ Transport Agency 2010a)
- passing lane spacing: distance in metres between the end of one passing lane (end of merge taper) and the start of the next passing lane (start of diverge taper)
- AADT: annual average daily traffic volume (in vehicles per day, two-way flow)
- percentage heavy vehicle (%HV): percentage of AADT which consists of heavy vehicles
- terrain: typical vertical profile along a route, classified as flat, rolling, or mountainous

- growth rate: percentage growth per annum in AADT volumes (developed separately for light and heavy vehicles)
- construction cost estimate: total cost estimate for building all passing lanes in a subsection (one direction of travel)
- maintenance cost estimate: on-going annual cost estimate for maintaining all passing lanes in a subsection (one direction of travel)
- cContour plot: plot presenting passing lane length and spacing combinations and associated BCR estimate
- optimal length/spacing: frequency and length of passing lanes required throughout a subsection to generate an optimal BCR without consideration of any major/minor network features.

A1.2 Required background knowledge

Basic working knowledge of the following concepts is assumed in order to apply this economic evaluation process.

- state highway function, volume and local section on-road detail (high level)
- passing lane design concepts as detailed in MoTSaM, section 3, figure 2.6, and chapters 2 and 4 in this report
- considerations for locating passing lane facilities in series through a subsection
- concepts of AADT, terrain and heavy vehicle composition
- economic principles such as BCRs, efficiency benefits, crash savings
- construction and maintenance cost estimates for passing lanes.

After applying the initial procedure, more comprehensive site construction cost estimates will need to be produced in order to refine the economic results.

A1.3 Process assumptions and limitations

The core process, application of the length/spacing BCR of contour plots, assumes the following:

- Input parameters (AADT, terrain, %HV, growth rate) are within a suitable range of the specified values (see section A3.3 for further guidance).
- The subsection identified is suitable for analysis (see section A3.2 for further guidance).

The process has the following limitations:

- It is based on the above AADT flows in typical rural locations. This includes the assumption of daily flow profiles and peak periods developed for rural locations. At higher volumes (20,000 – 25,000 AADT) the process naturally takes account of a level of peaking and tidal traffic flow. If the subsection is strongly influenced by tidal traffic flows (eg close to a major urban centre), or features high peaks in traffic volume (eg routes adjacent to large traffic generators), then consideration should be given to a site-specific investigation notably for AADTs less than 20,000.

- Depending on the subsection features and characteristics, the calculated BCR through the application of this process may not provide a comprehensive representation of on-road benefits. For example, the BCR is likely to be conservative if a flat/rolling subsection features a number of uphill sections where passing lanes can be located and aligned broadly with desirable spacings.

A1.4 2+1 schemes

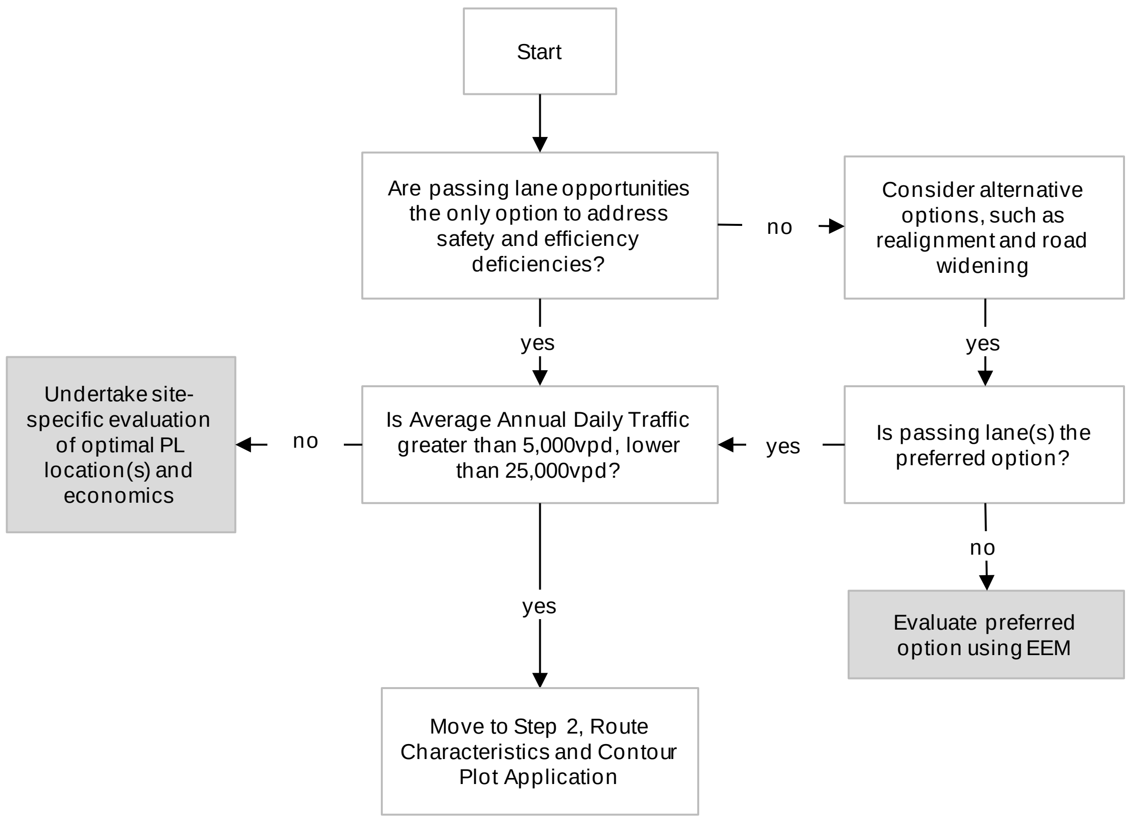
At higher volumes, it is likely that the optimal length and spacing arrangement will result in spacings which are similar to the passing lane lengths. Where the passing lane spacing is up to twice the passing lane length (eg a passing lane length of 1km and spacing of approximately 2km or less), a 2+1 scheme can be considered. A 2+1 scheme is essentially a carriageway with sufficient width for three continuous lanes of traffic, whereby the centre lane provides alternating passing opportunities for vehicles travelling in each directions (as opposed to the 'tack on' design which is commonly used for implementing passing lanes). If four-laning was a likely future requirement, construction of a continuous, parallel traffic lane (on one side the existing carriageway) and subsequent re-marking, is likely to be more efficient than separate construction activities occurring both sides of the existing carriageway.

In providing design guidance for 2+1 roadways, this report recommends the use of a wire rope barrier (WRB) on 2+1 roadways to provide physical separation between opposing traffic. It should be noted that the focus of this research is the merits of 2+1 configurations and the NZ Transport Agency has other specific research into different cross sections which will be used to the inform the design of 2+1 layouts. The WRB increases the implementation and maintenance costs of the scheme but generates a significant improvement in the severity of crashes. If a 2+1 scheme is proposed, this process should be applied by considering the passing lane length and spacing characteristics in a single direction of travel in the same way as passing lanes in series are assessed.

A2 Step 1: Determine the requirement for passing lanes

The requirement to apply passing lanes as a treatment on a rural state highway route should be checked before proceeding with defining subsections, and applying this process. Figure A.2 shows the process for determining the requirement for passing lanes.

Figure A.2 Identifying the requirement for passing lanes



A3 Step 2: Route characteristics and contour plot application

A3.1 Development of traffic subsection

The two-lane rural state highway route must be divided up into subsections with consistent characteristics. The definition of the subsection and the placement of passing lanes will be affected by major and minor network features.

Major network features dictate the need for separate subsections, and hence act as subsection borders. Minor network features affect the placement of passing lanes and are discussed in section A4.4. Major network features, ie subsection borders, include:

- significant changes in terrain, eg transition between flat/rolling and mountainous terrain
- urban areas
- major intersections
- speed limit changes to lower than 100km/h
- significant changes in AADT and/or %HV.

Figure A.3 shows an overview of the subsection analysis process.

Figure A.3 Subsection analysis overview

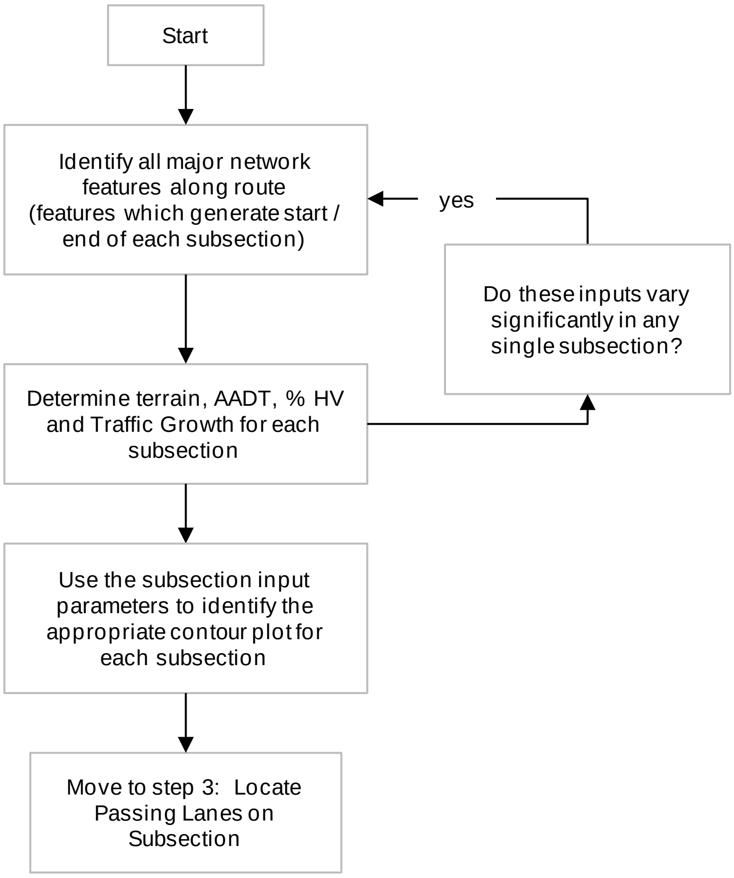
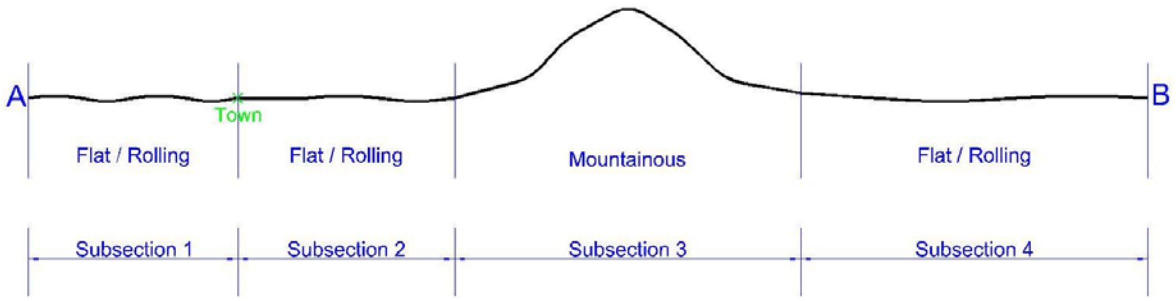


Figure A.4 shows an example of how a state highway route, between two major urban areas (A and B), may be divided into subsections for analysis.

Figure A.4 Example of the division of a state highway route into subsections



Passing lanes should only be provided once vehicles become sufficiently platooned. As the benefits of passing lanes continue some distance downstream of the passing facility, there is a minimum length for a subsection. These are approximately as follows:

- 5,000 AADT: 25km subsection
- 10,000 AADT: 15km subsection

- 15,000 AADT: 10km subsection
- 20,000 AADT: 8km subsection
- 25,000 AADT: 6km subsection.

If subsection lengths are less than those identified above, a site-specific evaluation may be required.

A3.2 Subsection suitability and assumptions

The following assumptions relating to the subsection need to apply to ensure an effective process:

- Subsection characteristics are reasonably generic and homogenous through the length of the subsection, eg similar AADT, %HV, gradient within each subsection (see table A.2 for criteria).
- There are no factors, subsection features, environmental issues etc, which would substantially alter vehicle behaviour from a typical New Zealand rural two-lane state highway function.
- The subsection does not feature extreme geometry, eg horizontal curves with advisory speeds of <65 through a flat/rolling section, <45 through a mountainous section.

A subsection that features existing passing lane facilities may be analysed. The focus would commonly be on whether infilling the route with additional passing lanes would offer suitable benefits. Accommodating existing passing lanes within this procedure is discussed further in section A4.3.1.

A3.3 Selection of appropriate contour plot

One or more subsections within the state highway route have now been identified, with common AADT, %HV, terrain and traffic growth characteristics specified for each subsection. These specific values can be used to identify the appropriate contour plot for each subsection. Table A.2 in section A7 provides the ranges for these inputs and identifies the appropriate contour plot to select.

A3.4 Apply contour plot – calculation of optimal length and spacing

The aim of the length and spacing contour plots is to identify the economically optimal length and spacing layout for passing lanes along a rural state highway for a given terrain, AADT, %HV and growth rate.

Once the appropriate contour plot has been identified, to apply the contour plot and begin the next step of locating passing lanes through the subsection, the first stage is simply to identify the 'sweet-spot' on the appropriate contour plot – roughly the mid-point of the highest BCR range on the x axis (spacing between passing lanes) and y axis (length of passing lanes).

For example, if the subsection has flat/rolling terrain, an AADT of 11,000 which includes 16%HV, 1% per annum light growth and 3% heavy growth, the selected contour plot would be: contour plot 7.2.2 – flat/rolling terrain, 10,000 AADT, medium %HV.

Using this contour plot, the sweet spot demonstrates that passing lanes of roughly 900m to 1,000m long, spaced around 9km to 10km apart, throughout the entire subsection generate the maximum subsection BCR range of 4.0 to 4.2.

The aim would be to locate passing lanes throughout the subsection at this length and frequency in order to maximise the efficiency of the scheme. The contour plot which is selected through this process will be used by the practitioner throughout the remainder of the process.

Note: The contour plot selected for this hypothetical subsection is used as a reference throughout the remaining steps of the process.

A4 Step 3: Locate passing lanes on subsection

A4.1 Overview

The optimal length and spacing determined from the contour plot in section A3.3 is used as a guide for placing passing lanes along the subsection. Passing lanes should be located at the point where vehicles become sufficiently platooned and in areas where alignment and forward visibility is considered safe. Figure A.5 outlines the process for locating the passing lanes through the subsection.

Figure A.5 Overview of locating passing lanes on subsection

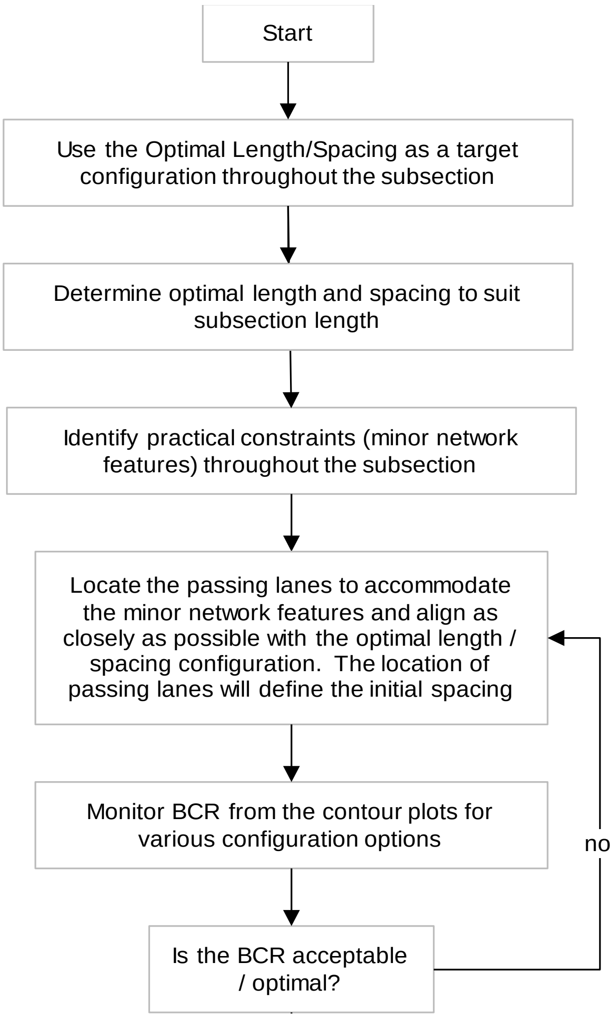
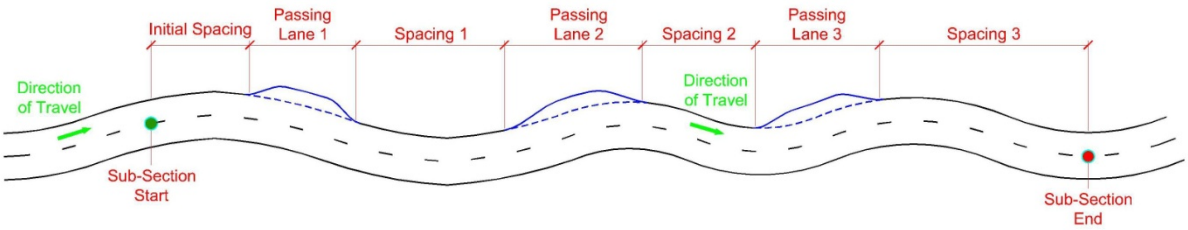


Figure A.6 provides a diagram of passing lane length and spacing placement through a fictitious subsection.

Figure A.6 Subsection passing lane locations – lengths and spacings



A4.2 Initial spacing

The major network feature (boundary feature) at the start of the subsection (eg town, urban speed limit, major intersection) will generally not generate high levels of platooning entering the subsection. In the examples listed, vehicles exiting a slower speed environment will tend to accelerate and travel for a period of time before beginning to form into platoons. Therefore the calculated optimal spacing between passing lanes can be used as a guide for the initial spacing, ie the length from the start of the subsection to the first passing lane.

If the major network feature at the start of the subsection does generate higher platoon levels, then the initial spacing may be shorter. The initial spacing can be confirmed through site investigations.

A4.3 Passing lane configuration

The optimal length/spacing configuration (as per the earlier example, 1,000m long, spaced at 9km) will only fit neatly within a subsection if:

- Subsection length = initial spacing + (multiple of [PL length + PL spacing])

In the majority of situations, the above equation will not hold true. Therefore, to place passing lanes within the subsection, the length and/or spacing will need to be varied. Assuming the optimal passing lane length is relatively fixed (as this feature physically enables the overtaking manoeuvres to occur), the most effective way to refine the design is to alter the passing lane spacings.

For example, an optimal length/spacing of 1,000m passing lanes located every 9km would not fit neatly within a 56km long subsection. Applying this configuration from the beginning of the subsection would result in:

- a 9km initial spacing
- 4 x 1,000m passing lanes with a 9km downstream spacing
- 1 x 1,000m passing lane with a 6km downstream spacing.

The short downstream section (6km rather than 9km) of the last passing lane means the benefits of that passing lane would not be fully realised. The appropriate design for the subsection would therefore have to be either:

- 5 x 1,000m passing lanes at 8.5km spacings, or
- 4 x 1,000m passing lanes at 10.4km spacings.

Referring to contour plot A7.3.3, it can be seen that the two options above generate BCRs of 4.0 to 4.2 and 3.8 to 4.0 (2012 values) respectively. This makes the five passing lanes at 8.5km spacings the most effective configuration.

A4.3.1 Accommodating existing passing lanes

This high-volume process is likely to be used to assess subsections with existing passing lanes. The principles used to determine the most effective length and spacing configuration (outlined above) can be applied to determine the effectiveness of infilling the route with additional passing lanes and the BCRs of these new installations.

For example, a 56km subsection discussed in the example above contains an existing 1,000m passing lane 21km into the subsection. The section up to the existing passing (21km) can be in-filled with an additional two passing lanes with spacings of 6.3km, or one passing lane with spacings of 10.0km. The 10.0km spacing is more effective, and these additional passing lanes would have a 2012 BCR of 4.0 to 4.2. The same process can be applied to the 34km section downstream of the existing passing lane, resulting in four additional passing lanes with spacings of 7.8km.

In the event that uncertainty exists about the most suitable passing lane length and spacing configuration for a given subsection, multiple designs can be individually and separately assessed in step 4 (section A5). Alternatively if the effects of existing passing lanes are more complex, a site-specific assessment may be carried out (see section A6).

A4.4 Locate the passing lanes within the subsection

When identifying specific locations for passing lanes within a subsection, a pragmatic approach is required to interpret the optimal length/spacing configuration against the characteristics of the subsection. For example, the optimal length/spacing may result in a passing lane being located where there is poor forward visibility and/or just before an uphill grade. In this situation it would be pragmatic to relax the length and spacing configuration and locate the passing lane on the uphill grade to maximise the number of overtaking manoeuvres that can occur, thereby maximising the benefits.

This balance of locating passing lanes requires knowledge and understanding of the operational characteristics of passing lanes. While not a definitive design guide, the following points/minor network features should be considered when selecting potential locations for passing lanes:

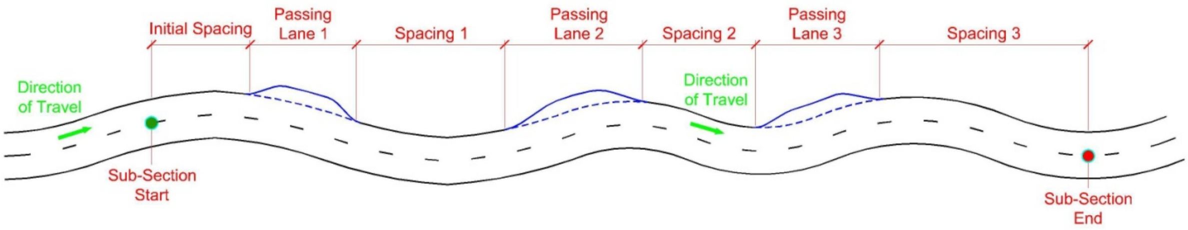
- Locate on uphill grades to generate maximum speed differential between heavy and light vehicles.
- Avoid bridges, unless a one-way bridge which is either very long and/or on a busy route.
- Avoid road network features such as culverts, no overtaking markings and minor intersections.
- Avoid and/or consider carefully the management/integration of accesses along the route.
- Consider forward visibility at the exit merge and termination of passing lane sites in the opposite direction.
- Avoid features which increase construction costs and/or land-take requirements.

A5 Step 4: Finalise subsection benefit-cost ratio

A5.1 Estimate BCR for facilities

The placement of passing lanes in the subsection resulting from the optimal length/spacing arrangement from step 3 will produce a design containing a series of passing lanes, each with varying lengths and spacings.

Figure A.7 Resulting length and spacing facilities



Starting from the first passing lane and spacing (see figure A.7), note the length and spacing for each facility. Refer back to the relevant contour plot and record the upper and lower BCR range for the relevant length/spacing for each facility. For example:

Table A.1 Example of BCRs for a series of passing lanes along subsection (taken from contour plot)

AADT	BCR from Chart	
	From	To
Passing lane 1 - 2	4.00	4.20
Passing lane 2 - 3	3.60	3.80
Passing lane 3 - end	3.80	4.00
Average	**3.80**	**4.00**

The above ranges would be averaged to produce the 2012 BCR range based on the contour plots for the subsection, in this case from 3.8 to 4.0.

Weighting the average BCR by the distance of each length/spacing feature is not appropriate as the BCR process already accounts for distance and this would result in 'double counting'.

If the calculated subsection BCR is not considered robust, or viable, then another passing lane design (spacing/length arrangement) may be attempted (go back to step 3, section A4). If this process fails to produce either a viable/robust BCR, or a clear optimal placement strategy, then consideration could be given to a site-specific investigation (see section A6) or alternative treatments. This is most notable if the BCR calculated is believed to be under or over-estimated due to specific on-road operation and network features. It is worth noting that passing lane strategies represent a reasonable roading investment in rural locations and the cost of a site-specific investigation is likely to fall within the margin of benefit-loss if passing lanes are placed in sub-optimal locations.

A5.2 Development construction cost estimate

The BCRs used in the contour plots are based on a consistent construction cost estimate of $1.0 million per kilometre (2012 value) of passing lane and incorporating any additional maintenance costs comparative to the do minimum (two-lane rural road carriageway) scenario. To finalise the BCR, a more robust construction cost estimate should be developed based on the passing lanes placed in the subsection. This should be completed by assessing the construction cost and yearly maintenance cost elements for each passing lane, in its on-road location, with particular emphasis on whether it differs significantly from $1.0 million per km (2012 value).

A5.3 Finalise BCR for subsection

To finalise the BCR for the passing lanes in the subsection, the average BCR calculated in 5.1 above (take the midpoint from the range) needs to be factored by the construction and maintenance cost estimate per kilometre of passing lane (and updated by appropriate cost update factors):

Final BCR = average BCR x (actual cost per km/$1million per km)

Having derived the final BCR (through an assessment of a site-specific cost estimate based on a specific passing lane(s) configuration), it may be lower or higher than anticipated due to specific on-road conditions. For example, the ability to locate passing lanes in a number of locations through the subsection on uphill grades may result in a clearly conservative BCR from the application of this procedure. Alternative designs may be tested through re-applying the process to differing length and spacing arrangements.

If the straight application of the process produces lower than anticipated returns, eg due to more effective placement on uphill gradients, or higher than anticipated returns, eg due to unique features of the subsection which would limit downstream benefits or passing opportunities within the length of passing facilities, then a site-specific investigation may be carried out. This would robustly determine the optimal location for passing lane facilities through the specific route in question and the economic returns.

A6 Site-specific assessment guide

A6.1 Transport modelling

A site-specific investigation into the optimal placement of passing lanes through a route, and estimation of the resulting BCR, is likely to require transport modelling. Broadly there are two types of transport models:

- Macroscopic models (also described as deterministic, strategic and regional) deal with aggregate traffic flows, eg hourly flows. SIDRA and SATURN are common examples.
- Microsimulation models are so called because they model individual vehicles in small time increments (commonly <1 second). Paramics, VISSIM and AIMSUN are examples.

Historically the simulation package TRARR has been used for investigations into passing lanes. The capabilities of this model may be limited compared with the above commercially developed software. Koorey (2003) noted a number of existing concerns and limitations have been identified with TRARR and no further upgrading is planned. In any respect, site-specific modelling should be calibrated to represent the existing on-road operation (do minimum) to a suitable level and capable of predicting the behaviours associated with the scheme being investigated. In the case of passing lanes, the key aspects are the dynamics of vehicles (speed distribution, acceleration etc) and representation of passing manoeuvres through the facility, resulting in travel time savings and the build-up and dispersion of platoons through the study area. Microsimulation models are the only tool with the flexibility and functionality to predict these outcomes and are therefore recommended for passing lane studies.

This section outlines, at a high level, guidelines on the recommended processes for microsimulation modelling of site-specific passing lane investigations.

A6.2 Software requirements

As well as the core vehicle behaviour distributions (ie target speed and target headway distributions), the microsimulation software selected for an investigation into passing lane economics will generally be required to include two elements not crucial in urban-style modelling:

- overtaking in the face of on-coming traffic (for representation of the do minimum)
- effects of gradients on (heavy) vehicles speeds and acceleration rates.

A6.3 Modelling process

The New Zealand Modelling User Group observed and modelled data comparison guideline[8] (currently being updated to more fully incorporate Transport Agency project work) contains target levels for transport model calibration and validation comparisons. Passing models are to some degree a unique situation, requiring some specific approaches, comparisons and techniques (notably, the representation of distributions of observed values has some importance) which are not directly covered by this guidance. However, it is anticipated that a passing model would comfortably meet the tightest count calibration criteria and, importantly, comfortably achieve the journey time comparison criteria (tighter levels are suggested below).

The following points outline, at a high level, the broad process and general considerations of a site-specific microsimulation modelling study measuring the economics of passing lane installation. Note, for the purposes of what is described below, calibration is assumed to be the adjustment of modelling parameters to represent the observed data as well as necessary to satisfy the objectives of the study, commonly an iterative process. Validation is assumed to be a comparison of modelled and independent observed data, ie a non-iterative process using data that has not been used in the calibration.

1. Study area: this should be carefully defined so as to be wide enough to capture the majority of benefits. There should be feature(s) at the start to influence platoon formation, and a sufficiently long section downstream of the scheme area to capture most of the travel time savings.
2. Field data: this will generally require daily traffic flows, vehicle composition data and travel times including intermediate points.
3. Additional field data: this can be helpful for obtaining speed and headway distribution data, eg from loops, or Transport Agency telemetric TMS sites, used to check that the distributions in the model software are appropriate (previous studies and non-local rural free-flow representative data may be appropriate).

8 www.ipenz.org.nz/ipenztg/Subgroups/NZMUGS/Documents/130913%20MUGS%20Model%20Data%20Comparison%20Criteria.docx

4 Vehicular data: this includes evidence of overtaking in the face of oncoming traffic, operation and utilisation of passing lanes, site gradients and response of vehicles to gradient affects.

5 Traffic flow definition: for economic calculations (annualisation) generally at least 12 hours of a typical day will need to be represented in the model, profiled in sub-hourly intervals (commonly 15 minutes or finer) by vehicle classification. Depending on the location, consideration may need to be given to representation of different day types for annualisation of economics, eg typical weekday, weekend, Fridays, long week peaks.

6 Vehicle definition: classification of vehicle types, notably the representation of various types of slower, heavy vehicles (eg semi-trailers, B-trains, coaches, towing vehicles).

7 Construction of road network: the modelled network is commonly straightforward, simple two-lane rural links. Consideration may need to be given to forward visibility for overtaking in the face of oncoming vehicles, gradients and horizontal curve design speeds.

8 Calibration: traffic flows are an input to this form of simple model and should be checked as a cross-reference, but should not be considered as a comprehensive calibration. Therefore calibration data may be limited, and consideration should be given to finding at least one source of calibration data, ideally speed and/or headway distribution data (see (3) above). Alternatively this could be overtaking percentages, or passing manoeuvres if the study area features existing passing lanes/areas of forward visibility (see (4) above).

9 Validation: ideally validation should be completed to observed journey times, sampled at intervals through the route. Modelled journey times should be within the greater of 30 seconds, or 10% compared with observed data for the complete route. The pattern of journey times through the sections route should broadly match those observed.

10 Forecasting: historical state highway traffic growth analysed from TMS data may be used to estimate annual forecast factors for light and heavy vehicles. Note: more recent trends (eg in the last five years) may need to be considered when developing forecasts from this method. A regional traffic model may be available.

11 Scheme model: the representation of passing lane facilities in the scheme test should be specified predictively in the model, not prescribed by the modeller, eg not set as simply a percentage of vehicles using the central (passing) lane.

12 Model application: model scenarios should be run a number of times so that all vehicles complete their journeys. Origin-destination statistics (eg travel time) should be calculated per run and the overall average across runs used in the economic calculations.

13 If using the S-Paramics microsimulation software, advice and guidance on parameters for use in rural two-lane modelling and passing lane assessments can be found in chapter 6.

A7 Spacing and efficiency lookups

Table A.2 provides a guide to selecting the appropriate contour plot for calculating the optimal length/spacing arrangement for the traffic subsection. The mid-point AADT range is used to demark the appropriate plot, ie AADT 5000, 10,000. The ranges given in table A.2 simply indicate the minimum/ maximum extent of the range around the midpoint, eg if the AADT through a specific subsection was recorded as 7,100vpd the AADT 5,000 charts would be used, if 17,800vpd was recorded the AADT 20,000 chart would be used. Similar ranges have been provided for the %HV and growth rates. The minimum AADT value to apply to this process has been specified as 4,000vpd as this is the generally accepted threshold for considering passing lanes.

Table A.2 Selection of length and spacing contour plot

Terrain	AADT	Heavy Vehicle %	Growth Rate	Plot
Flat / Rolling Terrain: Average gradient is <1%, maximum gradients around 4% (see EEM A7.5)	5,000 (4,000 - 7,500vpd)	Low 10 - 17%	0.0 -1.2% Lights, 0 - 5% Heavies	A.1.1
		Med 17 - 23.5%		A.1.2
		High 23.5 -30%		A.1.3
	10,000 (7,500 - 12,500vpd)	Low 8 - 13%	0.0 -1.2% Lights, 0 - 5% Heavies	A.2.1
		Med 13 - 17.5%		A.2.2
		High 17.5 - 22.5%		A.2.3
	15,000 (12,500 - 17,500vpd)	Low 8 - 12%	0 - 2% Lights, 0 - 5% Heavies	A.3.1
		Med 12 - 16%		A.3.2
		High 16 - 20%		A.3.3
	20,000 (17,500 - 22,500vpd)	Low 8 - 12%	0 - 3% Lights, 0 - 4% Heavies	A.4.1
		Med 12 - 16%		A.4.2
		High 16 - 20%		A.4.3
	25,000 (22,500 - 27,500vpd)	Low 6 - 9.5%	0 - 2% Lights, 0 - 4% Heavies	A.5.1
		Med 9.5 - 13%		A.5.2
		High 13 - 17%		A.5.3
Mountainous Terrain: Average gradient may vary, likely to be around 2-4%. More notably maximum gradient >6% (see EEM A7.5)	5,000 (2,500 - 7,500vpd)	Low 10 - 17%	0.0 -1.2% Lights, 0 - 5% Heavies	A.6.1
		Med 17 - 23.5%		A.6.2
		High 23.5 -30%		A.6.3
	10,000 (7,500 - 12,500vpd)	Low 8 - 13%	0.0 -1.2% Lights, 0 - 5% Heavies	A.7.1
		Med 13 - 17.5%		A.7.2
		High 17.5 - 22.5%		A.7.3
	15,000 (12,500 - 17,500vpd)	Low 8 - 12%	0 - 2% Lights, 0 - 5% Heavies	A.8.1
		Med 12 - 16%		A.8.2
		High 16 - 20%		A.8.3

The application of the contour plots and this method has been separated into two terrain types; flat/rolling and mountainous. Some interpolation between the results from these two terrain types may be required for 'hilly' terrain, which does not fall specifically into these categories (ie average gradient of 1% to 2%, maximum gradient of 4% to 6%). In such cases, passing lane placement is likely to follow a similar approach to mountainous terrain, eg placement of lanes on uphill sections. A site-specific assessment may be required where no obvious terrain distinction and/or lane placement can be established.

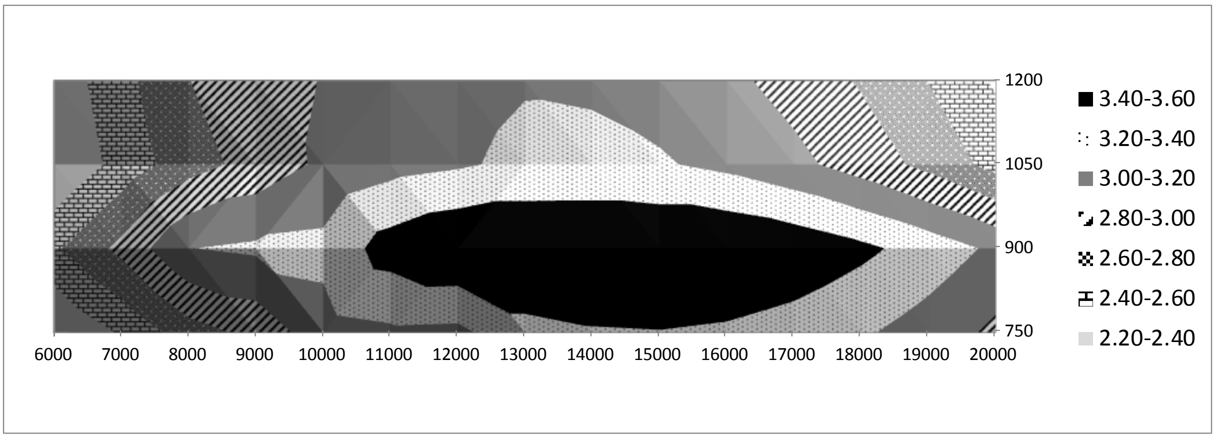
The 'contour plots' discussed throughout this process are provided in section A7.1 onwards. The plots are set out as follows:

- x axis: passing lane spacing (metres)
- y axis: passing lane length (metres)
- contour bands: BCR in 0.20 increments (2012 values).

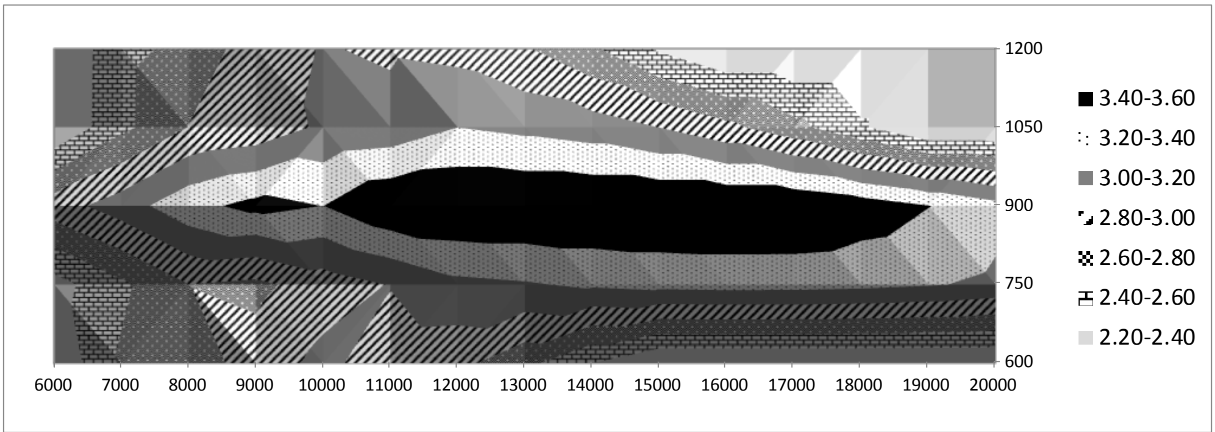
Once the specific contour plot relating to the subsection has been identified from the parameters contained in table A.2 above, the BCR values for various spacing and length arrangements can be read off the plot.

A7.1 Flat/rolling terrain, AADT 5,000

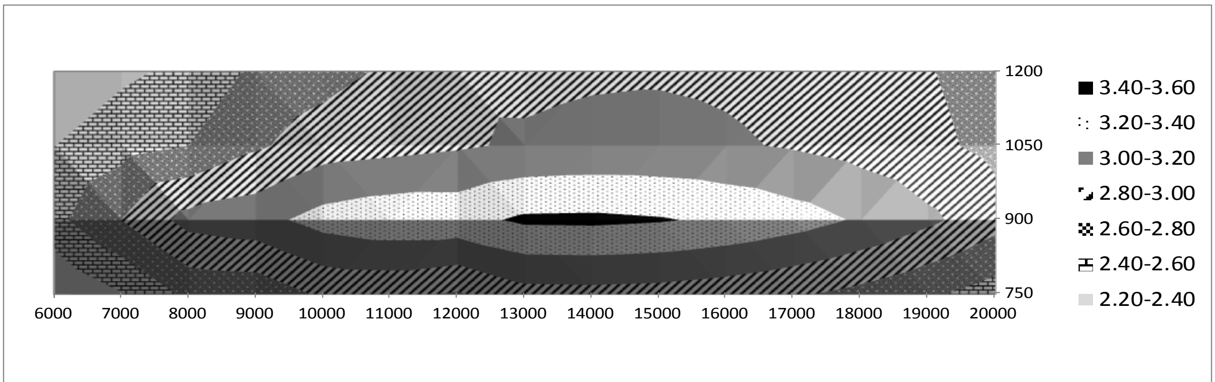
A7.1.1 Low heavy vehicle percentage



A7.1.2 Medium heavy vehicle percentage

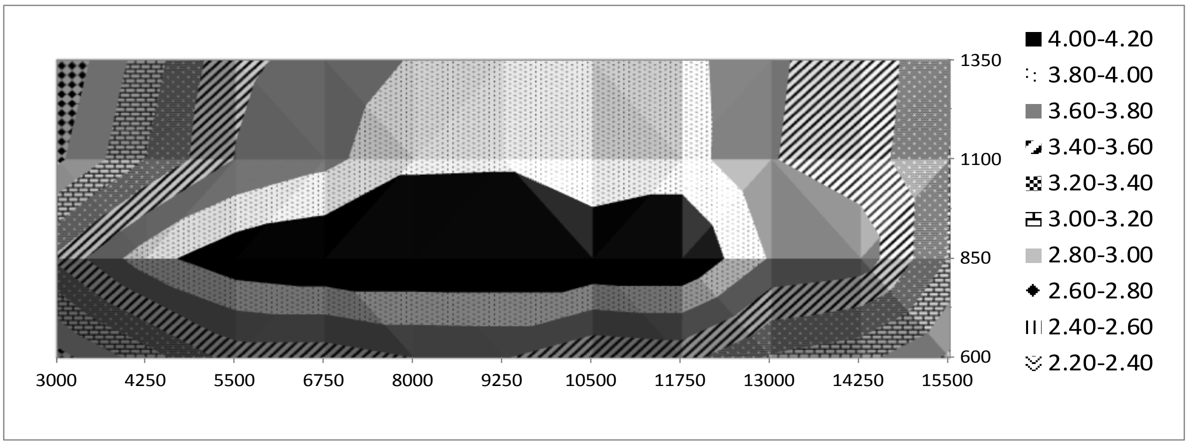


A7.1.3 High heavy vehicle percentage

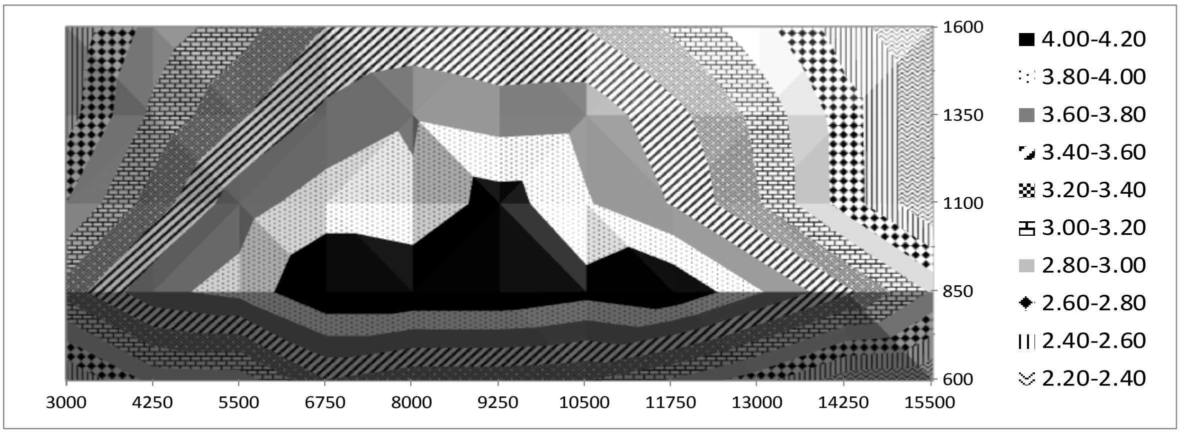


A7.2 Flat/rolling terrain, AADT 10,000

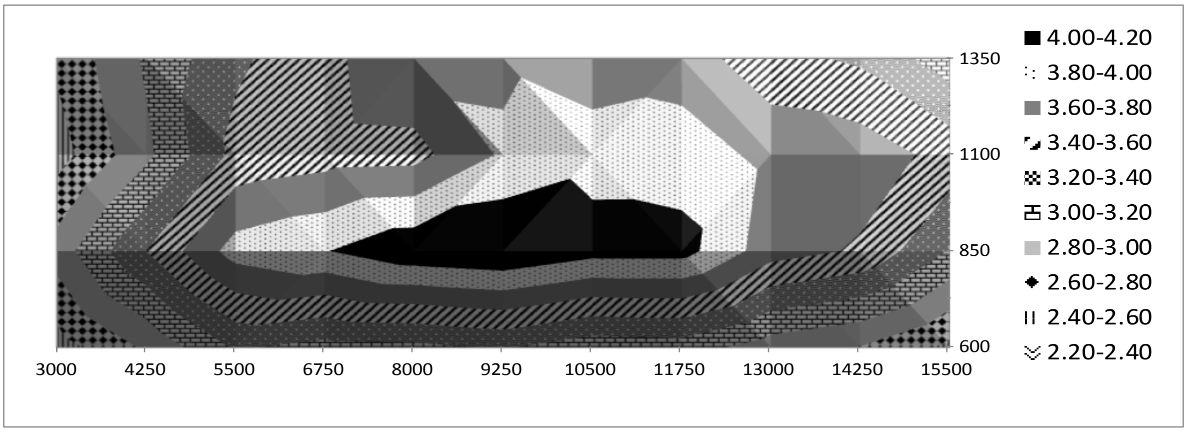
A7.2.1 Low heavy vehicle percentage



A7.2.2 Medium heavy vehicle percentage

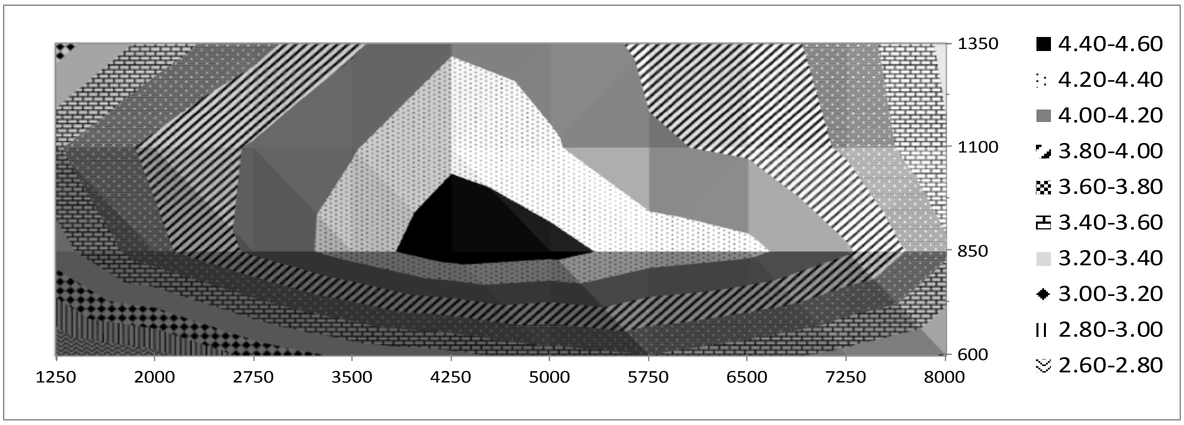


A7.2.3 High heavy vehicle percentage

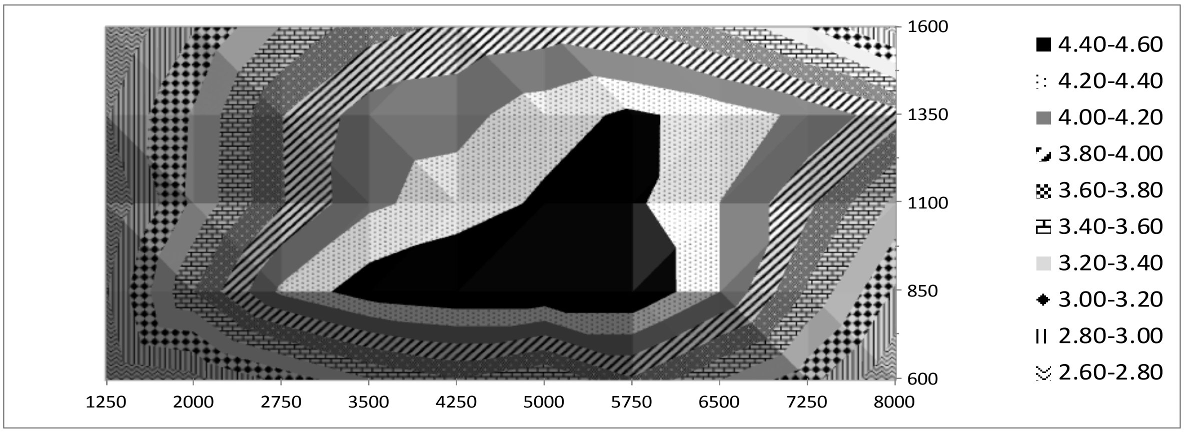


A7.3 Flat/rolling terrain, AADT 15,000

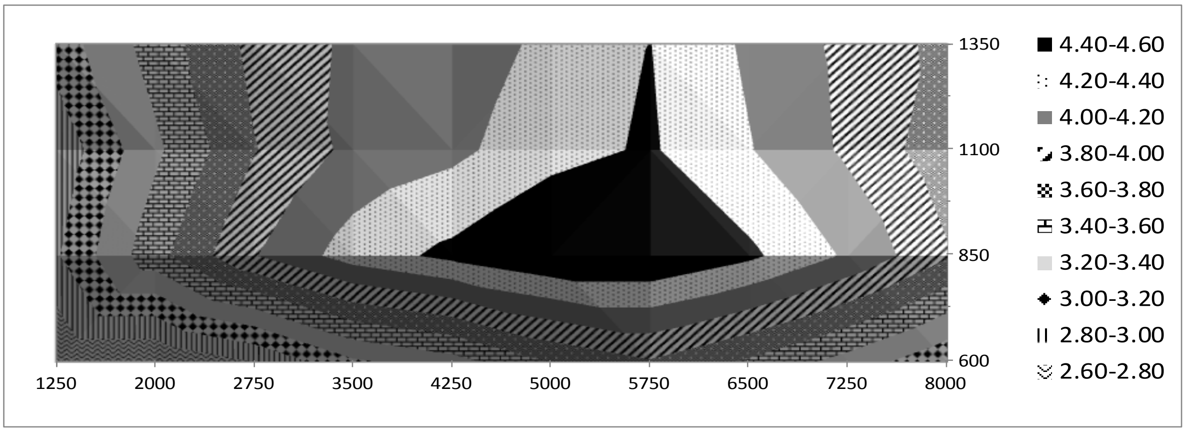
A7.3.1 Low heavy vehicle percentage



A7.3.2 Medium heavy vehicle percentage

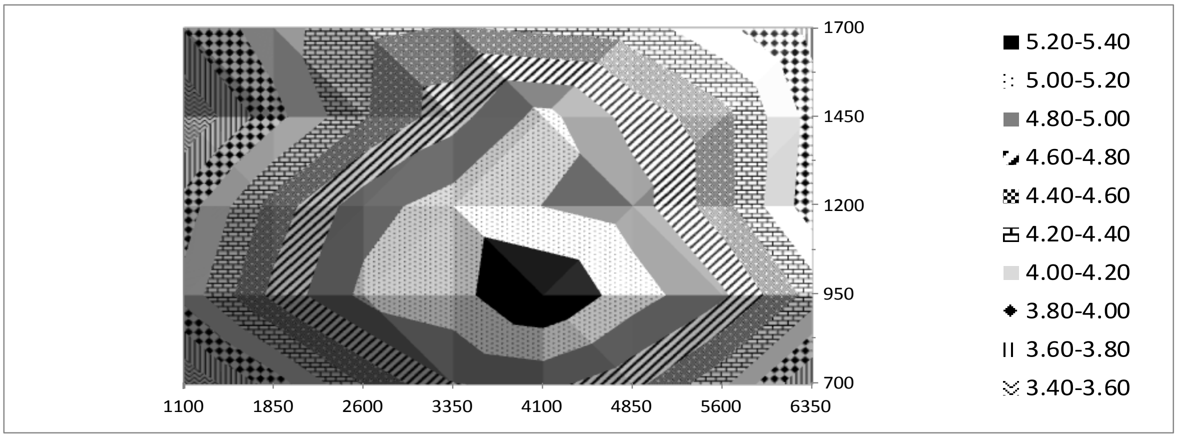


A7.3.3 High heavy vehicle percentage

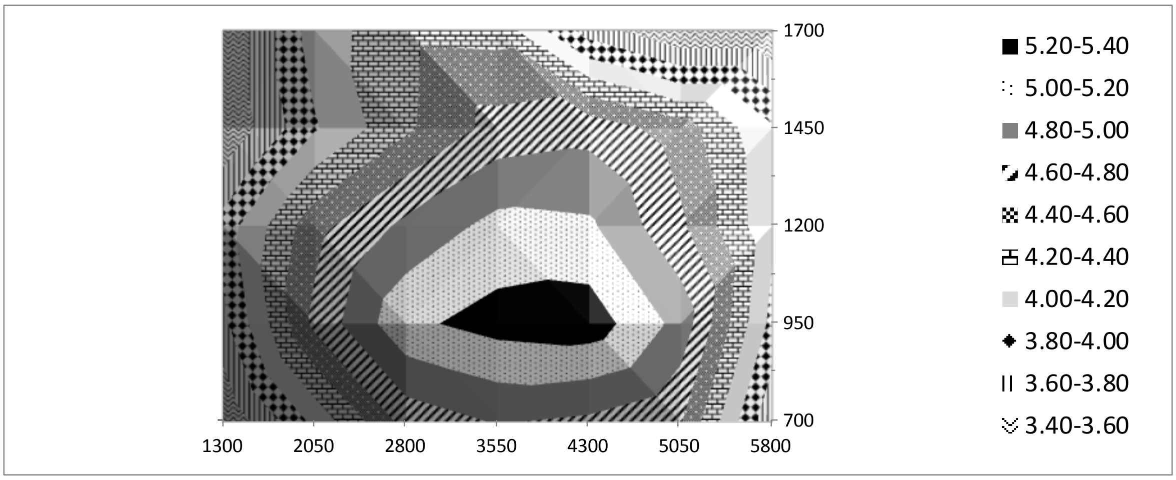


A7.4 Flat/rolling terrain, AADT 20,000

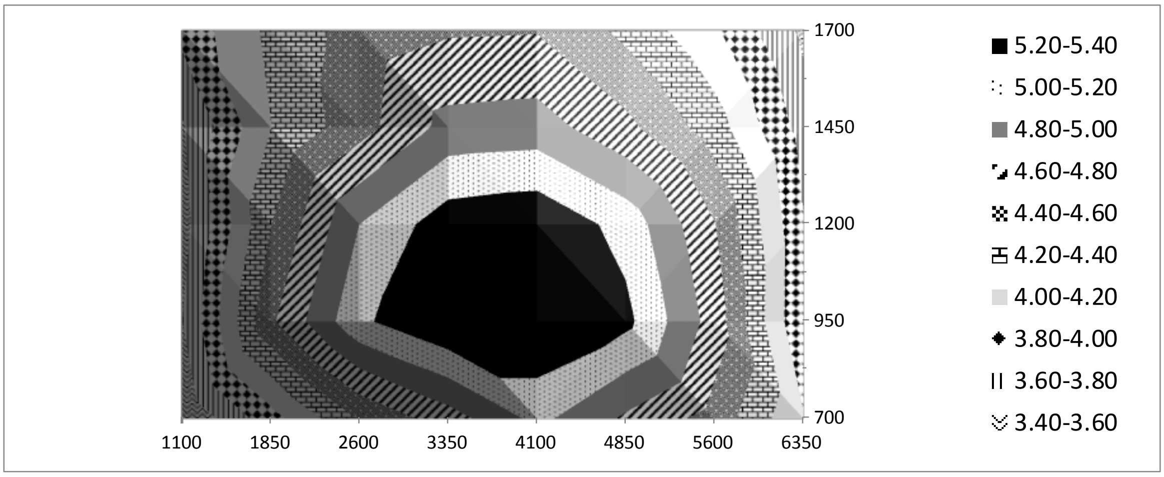
A7.4.1 Low heavy vehicle percentage



A7.4.2 Medium heavy vehicle percentage

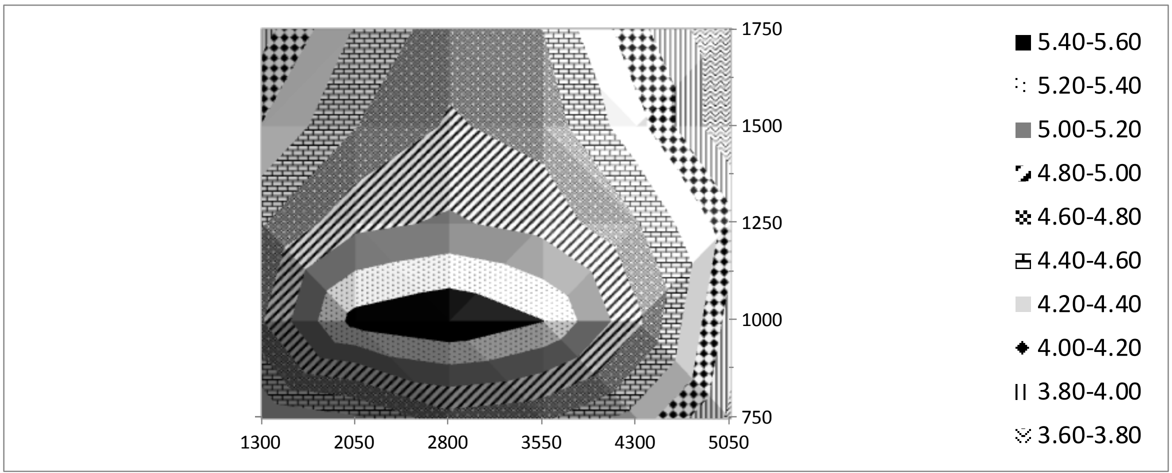


A7.4.3 High heavy vehicle percentage

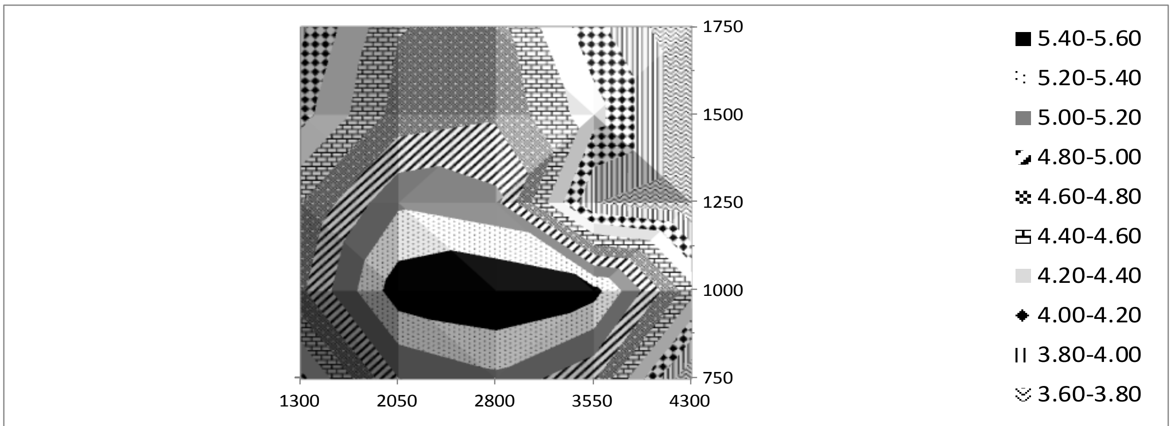


A7.5 Flat/rolling terrain, AADT 25,000

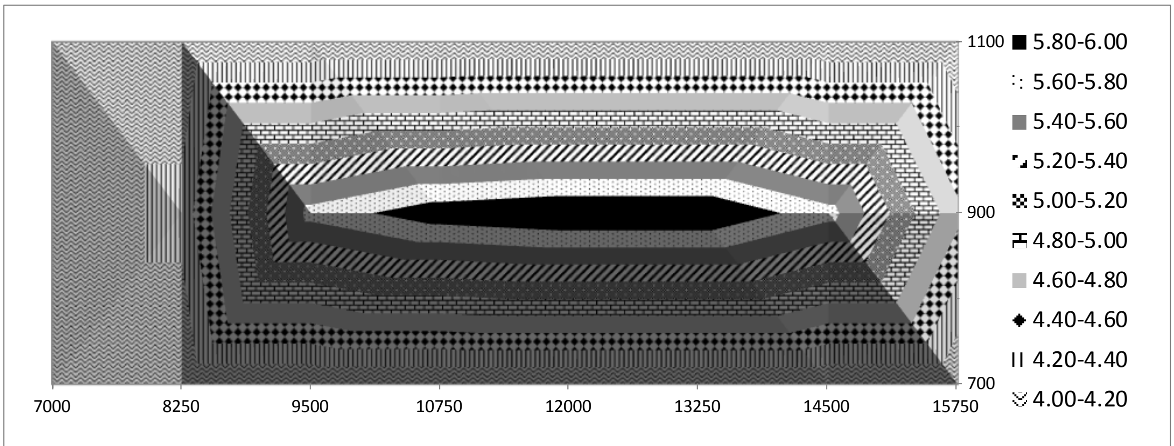
A7.5.1 Low heavy vehicle percentage



A7.5.2 Medium heavy vehicle percentage

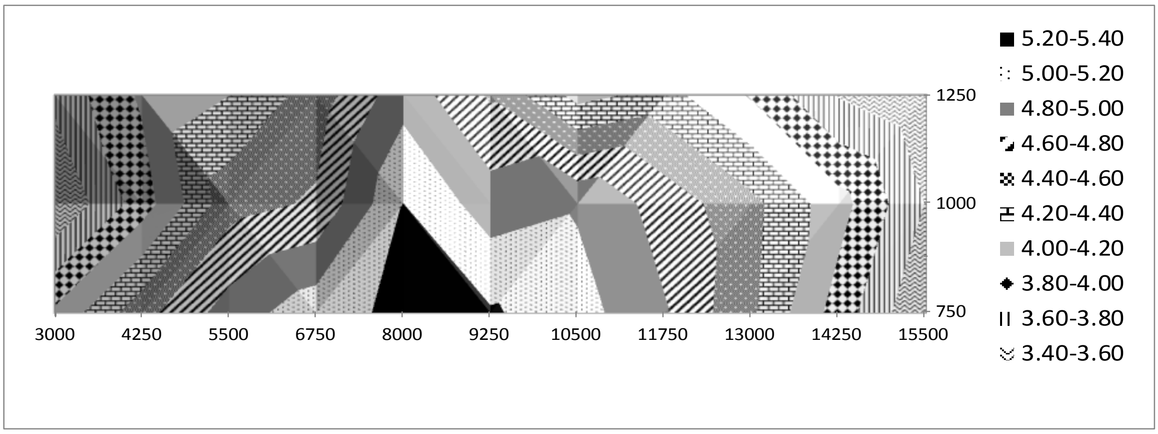


A7.5.3 High heavy vehicle percentage

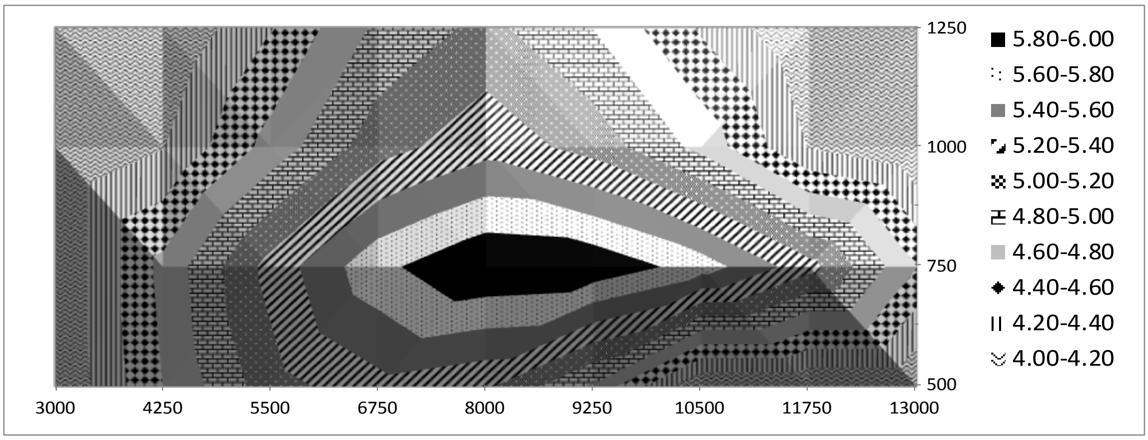


A7.6 Mountainous terrain, AADT 10,000

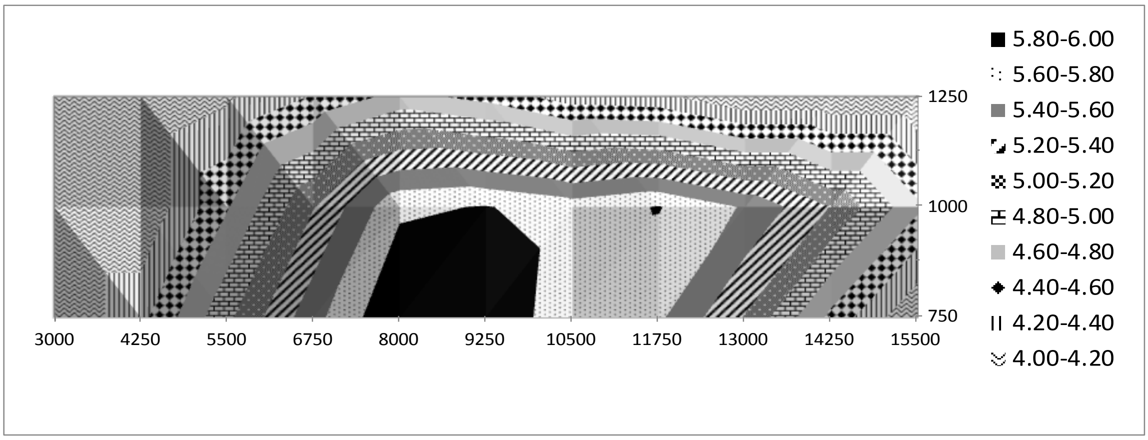
A7.6.1 Low heavy vehicle percentage



A7.6.2 Medium heavy vehicle percentage

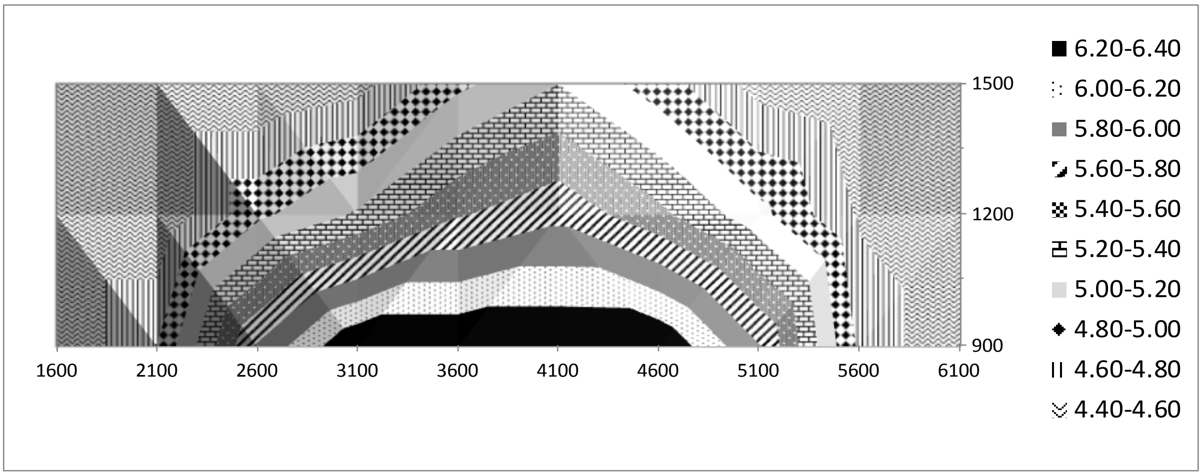


A7.6.3 High heavy vehicle percentage

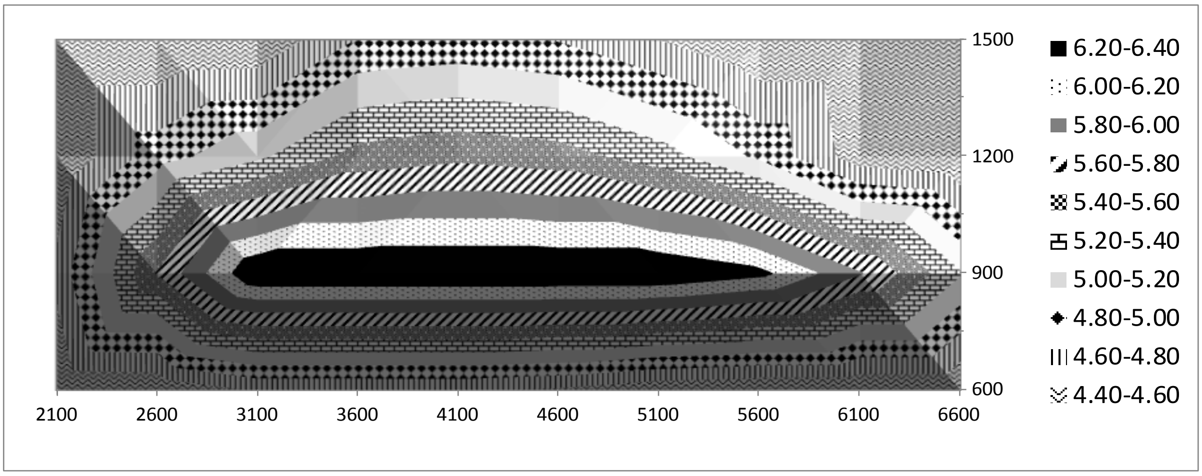


A7.7 Mountainous terrain, AADT 15,000

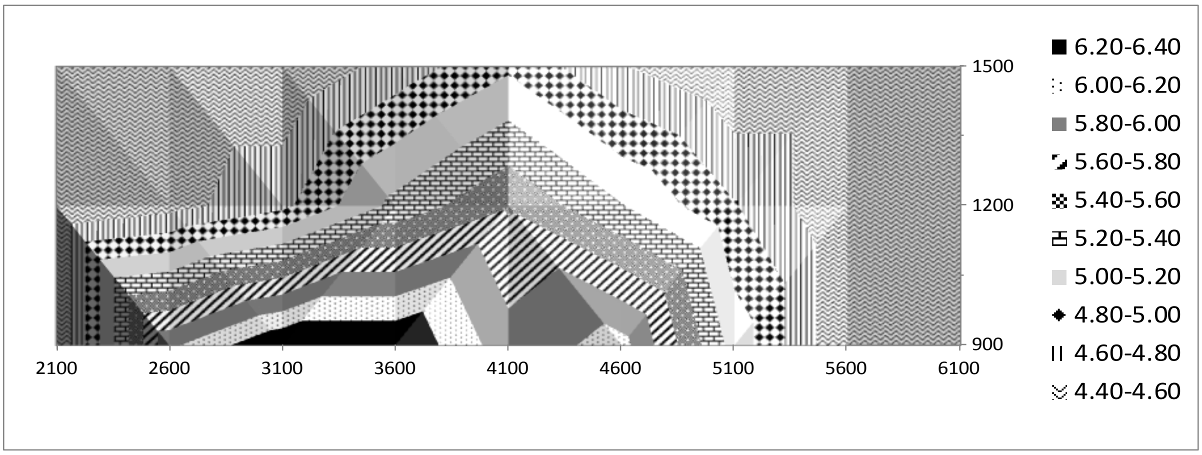
A7.7.1 Low heavy vehicle percentage



A7.7.2 Medium heavy vehicle percentage

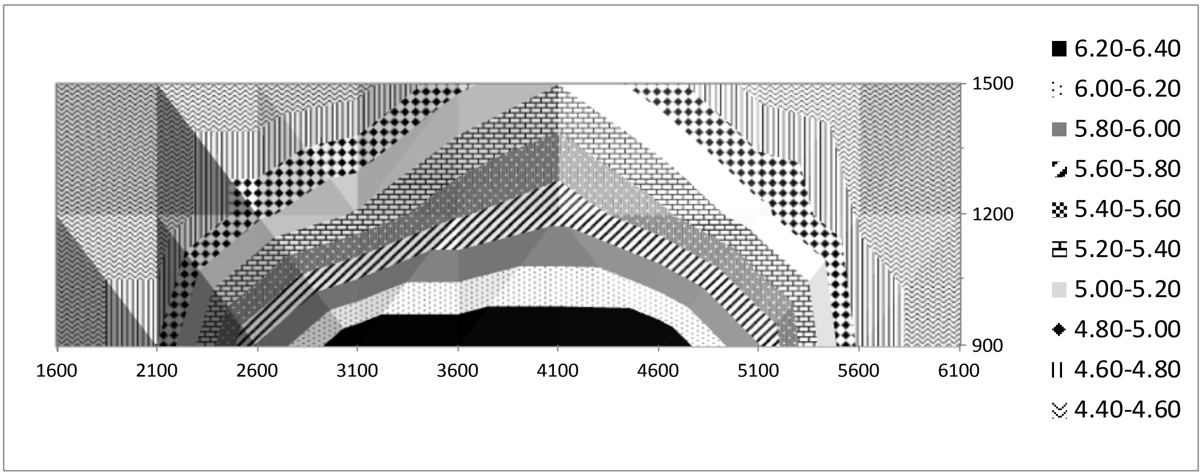


A7.7.3 High heavy vehicle percentage

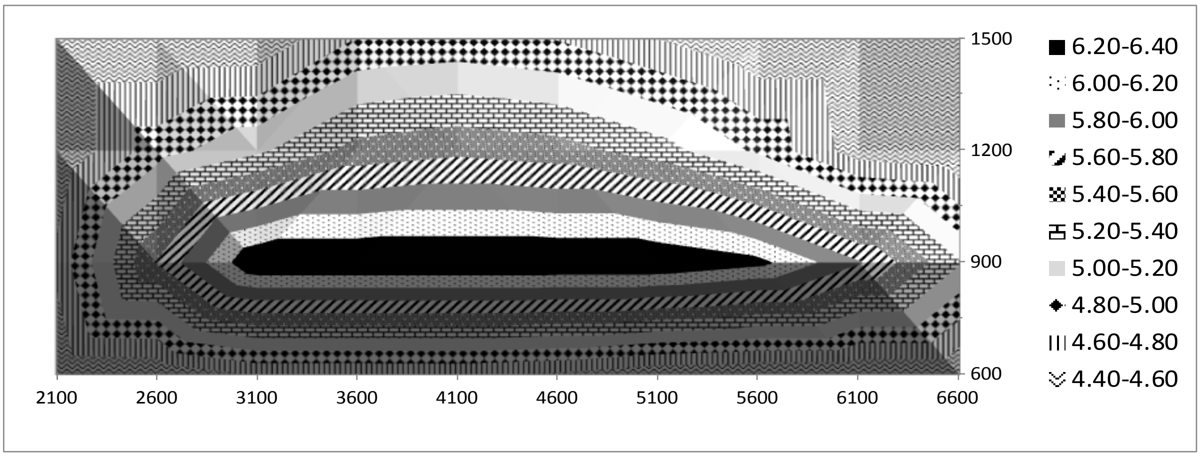


A7.8 Mountainous terrain, AADT 15,000

A7.8.1 Low heavy vehicle percentage



A7.8.2 Medium heavy vehicle percentage



A7.8.3 High heavy vehicle percentage