*Central Laboratories Report 08- 529B05.00*

# Operational Evaluation of Representative Passing Lanes Against Proposed Guidelines

**P.D.Cenek**

**T.J.Lester**

**Prepared for:**

NZ TRANSPORT AGENCY
WAKA KOTAHI

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**October 2008**

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## Executive Summary


Automated traffic surveys using pneumatic tube vehicle classifiers were conducted at five passing lanes and one slow vehicle bay. The surveys were structured to determine the impact of passing facilities on traffic operations to allow the NZ Transport Agency's proposed strategy for passing and overtaking treatments to be independently evaluated. Three key locations were simultaneously surveyed: up to 2 km upstream of the passing facility, within the passing facility, and for up to 12 km downstream of the passing facility.

The acquired data covered passing lengths from 0.325 km to 1.397 km, hourly flows up to 800 vehicles per hour, percentage heavy vehicles from 5% to 22%, average gradients along the passing facility from 0.3% to 7.2%, and upstream percentage of vehicles delayed in platoons up to 71%.

The surveys were conducted over the period 10th July 2007 to 27th July 2007, each survey period at a passing facility lasting a minimum of 72 hours, with the monitoring period covering 3 full week days from Monday to Friday inclusive, so that a large proportion of high hourly flows were captured. The vehicle classifiers were used to record traffic volumes, vehicle classes, speeds and headways (vehicle spacings).

The measures of effectiveness employed in the study were:

- percentage of vehicles following, which relates to passing lane spacing
- passing rate, which relates to passing activity at a location along the passing facility
- normalised passing rate, which relates passing activity to the length of the passing facility
- percentage of vehicles passing, which relates passing activity as a proportion of one-way flow in the same direction.

The principal finding was that the NZ Transport Agency's proposed Policy framework for passing and overtaking treatments has passing facility lengths and spacings reasonably correct. The Policy framework also appears to have a degree of "future proofing" in the passing lane length to take into account expected increases in heavy commercial traffic, which will require passing facilities to be longer to maintain their effectiveness. The structure of the Policy framework, which is based around traffic flow and road gradient, was also shown to be correct as these two parameters significantly influenced operational effectiveness.

Secondary findings considered important were:

1. Crawler shoulders at lower traffic volumes and crawler lanes at higher traffic volumes appear to have been omitted as treatments in the long-term framework but are allowed for within the NZ Transport Agency's Passing and Overtaking Policy. The survey results indicate that these treatments could be included in the long-term framework for mountainous road gradients to provide consistency with other parts of the NZ Transport Agency's Passing and Overtaking Policy.

2. For a specified passing facility length, the percentage passing increases with increasing one-way flow irrespective of road gradient. For one-way flows below 200 vph, there is an indication that there is more passing activity on passing facilities in mountainous and rolling road

gradient. For one-way flows greater than 400 vph, passing facilities have more passing activity as they become longer, irrespective of road gradient.

3. Per kilometre of facility, the most effective with respect to passing rates are slow vehicle bays, followed by short passing lanes, with long passing lanes being the least effective. This suggests that more short passing lanes would be more effective than fewer long passing lanes. However, treatments with shorter passing lengths have less traffic flow capacity and so their service life is limited. Therefore, shorter treatments are only suitable over lower traffic ranges.

4. The passing rate was shown to increase with increasing flow. Up to 200-300 vehicles per hour, the passing rate is fairly constant throughout the length of the passing facility. Above this flow rate, the highest passing rates occur near the middle of the facility for short passing lanes and a quarter of the way down the facility for long passing lanes.

5. Immediately downstream of the passing facility, the reduction in percentage of following vehicles based on a 4-seconds headway criterion was 4.4 percent. However, there was an indication that the difference in percentage of following vehicles upstream and downstream of the passing facility reduces with increasing flow for both 2 and 4-seconds headway. This merits further investigation as many factors could cause this situation such as percentage of following as a function of traffic flow and downstream conditions near to the passing lane taper.

6. The downstream operational length of passing lanes decreases with increasing traffic volume and increasing headway. Typically, for the same hourly traffic flow, the downstream operational length derived for a 2-seconds headway is between 1.1 and 2 times that calculated for a 4 second headway.

7. "Across centreline" passing rates observed where passing in the opposite direction at a passing facility is permitted was minimal at 0.8% to 1.2% corresponding to 3 and 7 passes/hour/km for a peak hourly flow of 350 vehicles per hour. As this is significantly lower than expected from overseas research, further investigation is merited to establish if the cause is either site characteristics or safety concerns or unfamiliarity with road rules.

8. When applied to the surveyed passing facilities, overseas models overestimated their operational effectiveness in terms of passing rates and reduction in the percentage of following vehicles. This highlights the need to calibrate overseas derived models for local conditions.

9. Regression modelling was applied to operational data acquired over a 72 hour period at each of the six sites surveyed. Traffic flows up to 808 vph were covered. The regression modelling showed operational effectiveness of a passing facility to be strongly related to traffic flow, road gradient in the vicinity of the passing facility, and percentage of light vehicles towing and heavy commercial vehicles in the traffic stream. Of all the variables investigated, passing related measures, such as percentage of vehicles passing and normalised passing rate, appeared to provide the most robust measure of operational effectiveness of passing lanes and so their use is recommended in any further studies of passing facilities

10. Given the quality of the database that has been generated, it is recommended that additional analyses involving horizontal and vertical sight distances and vehicle speeds should be undertaken to better explain the variances observed in the operational effectiveness of the six passing facilities surveyed.

11. Lower average percentage of passing vehicles was observed at sites 3e (short passing lane in mountainous road gradient) and 4j (long passing lane in flat road gradient). Accordingly, a limited investigation was undertaken to determine possible reasons for the observation. For site 3e , a 1.4 km long straight with good visibility ending about 1.5 km upstream of the site with another 300 m of clear sight distance before the passing lane diverge was attributed as the most likely reason. For site 4j, its regular targeting by NZ Police for mobile speed enforcement was attributed as the most likely reason. Both these features made sites 3e and 4j less than ideal for evaluating the NZ Transport Agency's proposed Policy framework for passing and overtaking treatments.

12. Additional sites could be investigated to verify the Policy framework over a greater range of traffic flows and road gradients and to improve the robustness of mathematical models derived for predicting the operational effectiveness of passing facilities.

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# 1 Introduction

As part of the NZ Transport Agency's Passing and Overtaking Policy, a long-term framework for passing and overtaking treatments has been prepared for application to the two-lane state highway network in rural and peri-urban areas (Transit, 2007). Part of this framework relates the length and frequency of slow vehicle bays (SVBs) and passing lanes (PLs) to projected traffic volumes and road gradient. This long-term framework is summarised in Table 1 for ready reference.

As the proposed lengths and frequencies for SVBs and PLs have been derived mainly from overseas research in conjunction with some New Zealand research, operational field data was collected using traffic counters at six existing passing sites that represented various treatments suggested in Table 1. The survey data acquired was analysed to confirm the appropriateness of the long-term framework and quantify its ability to improve traffic service on two-lane state highways.

The six sites covered passing lanes with passing either prohibited or permitted in the opposing direction, as shown in Figure 1, to establish if there is a significant operational advantage through permitting passing in the untreated direction that should be accounted for in economic assessments of passing treatments.

Besides summarising the principal results of the field measurements in relation to the NZ Transport Agency's proposed long-term framework for passing and overtaking treatments, this report includes:

- a detailed description of the six sites selected for investigation;
- the field data collection plan adopted and associated processing of the traffic count data acquired to obtain measures of operational effectiveness;
- comparisons between observed traffic behaviour and that predicted using models derived from operational data acquired at the selected six sites, which in most cases satisfied requirements of the long-term framework; and
- recommendations as to how the long-term framework could be improved.

**Table 1: NZ Transport Agency's proposed long-term (25-30 years) framework for passing and overtaking treatments**

<table>
<tr><th rowspan="3">Projected AADT<br>(vpd)</th><th colspan="3">Suggested Passing and Overtaking Treatments</th></tr>
<tr><th colspan="3">Road Gradient</th></tr>
<tr><th>Flat</th><th>Rolling</th><th>Mountainous</th></tr>
<tr><td>0 - 2000</td><td colspan="3">Overtaking (OT)<br>(OT sight distance improvements, OT enhancements,<br>possible isolated shoulder widening/crawler shoulder/ SVB[1]/ short PL)</td></tr>
<tr><td>2000 - 4000</td><td>Overtaking<br>(as above)</td><td colspan="2">Mainly OT, possibly some SVB[1]/short PL @ 10 km</td></tr>
<tr><td>4000 - 5000<br>(General transition to PL's)</td><td>Mainly OT, possibly some SVB[1]/ short PL @ 10 km</td><td>PL @ 5-10 km,<br>1.2 km + tapers, &<br>OT enhancements</td><td rowspan="2">PL @ 5 km,<br>1 km + tapers, & possible OT enhancements</td></tr>
<tr><td>5000 - 7000</td><td colspan="2">PL @ 5 or 10 km[2], 1.2 km + tapers, &<br>OT enhancements</td></tr>
<tr><td>7000 - 10000</td><td colspan="2">PL @ 5 or 10 km[2], 1.5 km + tapers, & OT enhancements</td><td>PL @ 5 km,<br>1.2 km + tapers, &<br>possible<br>OT enhancements</td></tr>
<tr><td>10000 - 12000<br>(General transition to 2+1 lanes)[3,4]</td><td>PL @ 5 km, 1.5 km + tapers, & possible OT enhancements</td><td>2+1 lanes<br>(subject to 4-lane comparison)</td><td rowspan="3">PL @ 5 km,<br>1.2 km - 1.5 km+ tapers</td></tr>
<tr><td>12000 - 20000</td><td colspan="2">2+1 lanes (subject to 4-lane comparison)</td></tr>
<tr><td>20000 -25000<br>(General transition to 4 lanes)</td><td colspan="2">Mixed 2+1 lanes & 4 - lanes</td></tr>
</table>

| **KEY:** | **Overtaking** | **Mainly Overtaking** | **Passing & Overtaking** | **Passing** |
|---|---|---|---|---|

<table>
<tr><td>Abbreviations:</td><td>SVB - slow vehicle bay<br>PL - passing lane. Short passing lane defined as 600m – 800m long + tapers.<br>OT - overtaking</td></tr>
<tr><td>Notes:</td><td>1. Where appropriate, a SVB is able to be easily altered to a short PL or PL at a later date.<br>2. Along the same road section a mixed layout with 5 km spacings in higher demand locations and 10 km spacings in lower demand locations.<br>3. For flat or rolling road gradient, the combination of passing length and spacing may not be sufficient to dissipate vehicle platoons 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.<br>4. 10,000-25,000 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.</td></tr>
</table>

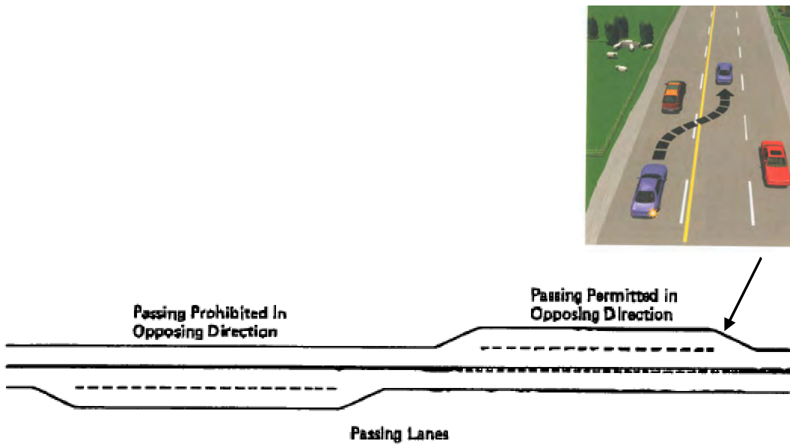


**Figure 1: Passing lane configurations investigated**

## 2 Terminology

The following terminology has been adopted in this report to ensure consistency with previous work, such as Harwood et al (1985) and Koorey and Gu (2001), which along with other New Zealand and overseas research have formed the basis of the NZ Transport Agency's Passing and Overtaking Policy.

| | |
|---|---|
| ***ADT*** | Average daily traffic flow (usually taken over 7 days of the week) |
| ***AADT*** | For telemetry sites, annual average daily traffic is calculated by counting the number of vehicles passing a roadside observation point in a year and dividing this number by 365. (Where locations are surveyed for 1-4 weeks of the year, ADT values are factored (annualised) to approximate a typical daily traffic flow.) |
| ***Bunching*** | Grouping of vehicles in the same direction with restricted speed caused by a slow moving vehicle at the head of the bunch and limited overtaking opportunities. Calculated as the ratio of following vehicles to total vehicles and normally expressed as a percentage. |
| ***Desired Speed*** | The speed that drivers would like to travel when not constrained by other traffic. This is largely dependent on the road alignment. Also known as *free speed* or *unimpeded* speed. |
| ***Diverge Area*** | Zone at the start of the *passing lane* where one lane tapers into two. Also known as *lane addition taper*. |
| ***Effective Downstream Operational Length*** | Distance downstream of a passing facility at which the level of *bunching* reaches the same level as it was immediately prior to the passing facility. Corresponds to the situation where the majority of the vehicles have rebunched after the passing lane. |
| ***Following Vehicles*** | Vehicles that are sufficiently close to the vehicle in front to be affected by the speed of the front vehicle. Vehicle with *headways* of 4 seconds or less are considered to be following. |
| ***Free Vehicles*** | Vehicles able to travel at their *desired speed*. This includes vehicles on their own, i.e. not part of a multi-vehicle *platoon*, and *leading vehicles*. Vehicles with *headways* of more than 4 seconds are usually considered to be free. |
| ***Headway*** | The amount of separation between successive vehicles. Can be measured either by distance or time. Usually measured from the front of one vehicle to the front of the next. |
| ***Inner Lane*** | For two or more lanes in the same direction, lane closest to the centreline. |

| | |
|---|---|
| ***Leading Vehicles*** | The vehicle at the head of a multi-vehicle *platoon*. Leading vehicles are able to travel at their *desired speed*. |
| ***Merge Area*** | Zone at the end of the *passing lane* where the two lanes taper into one. Also known as *lane drop taper*. |
| ***Normalised Passing Rate*** | Number of completed passes per hour per kilometre in one direction of travel. |
| ***Outer Lane*** | For two or more lanes in the same direction, lane closest to the edge of the seal. |
| ***Overtaking*** | Within the context of the NZ Transport Agency's Passing and Overtaking Policy, a vehicle crosses the centreline into the opposing traffic lane to pass slower vehicles travelling in the same direction. |
| ***Overtaking Distance*** | Distance required for one vehicle to overtake another vehicle. |
| ***Overtaking Sight Distance*** | The sight distance required for a driver to initiate and safely complete an overtaking manoeuvre. |
| ***Passing Lane*** | An auxiliary lane provided to allow for slower moving vehicles to be passed. It is line marked so that all traffic is initially directed into the left hand lane, with the inner lane (closest to the centreline) being used to pass. For the purposes of this report, the passing length of the passing lane does not include diverge and merge areas. |
| ***Passing Lane Spacing*** | Distance from end of the upstream passing lane's merge taper to the start of the downstream passing lane's diverge taper. |
| ***Passing Rate*** | Number of completed passes per hour in one direction of travel. |
| ***Percentage of Following Vehicles*** | Ratio of following vehicles to total vehicles, normally expressed as a percentage. |
| ***Percentage of Passing Vehicles*** | Ratio of the number of passes to one-way flow at a fixed location within the passing treatment, expressed as a percentage |
| ***Platoon*** | A group of vehicles clustered together (i.e. small headways) and all travelling at approximately the same speed as the leading vehicle. Also known as queues or bunches. The size of the platoon is defined by the number of vehicles. A vehicle on its own is considered a platoon of size one. |
| ***Sight Distance*** | The road distance ahead of the driver that is visible. This enables the driver to assess whether it is safe to pass. Refer to Austroads (2003) "Rural Road Design" for further information, especially with regard to object and eye heights. |

***Slow Vehicle Bay*** A very short auxiliary lane (of the order of up to 300 m long in New Zealand) that allows a slow vehicle to pull aside to allow a following vehicle to pass. Slow vehicles have to give way to the main traffic flow at the end of the bay.

***Through Lane*** For slow vehicle bays, the through lane is the lane closest to the centreline (inner lane) whereas for passing lanes it is the lane closest to the kerb (outer lane) when considering traffic flows in the treated direction.

# 3 Study Sites

Within cost and time constraints, it was initially considered that the most expedient way to evaluate the proposed long-term framework would be to select two study passing sites for each type of road gradient (i.e. flat, rolling and mountainous) with current traffic volumes that covered transitions from (1) SVBs/short PLs to medium PLs and (2) long PLs to 2+1 lanes as shown in Table 1. A critical requirement for each study passing site was that its length had to conform to the length specified in Table 1 for the appropriate road gradient and projected traffic volume.

Additional desired attributes of the study sites were as follows:

- Close proximity to the supplier of traffic counting services to minimise travelling costs.
- Highly variable traffic volume and composition (percentage of light vehicle towing (LVT) and heavy commercial vehicles (HCV)) during the course of the day to allow a range of flow rates and bunching to be investigated.
- The 2 km's before and the 10 to 15 km's after the passing facility should be free of major side roads, one-lane bridges, railway crossings, road works and away from major settlements. In addition, the passing facility should be free of turning bays and egress points to properties. These attributes were considered necessary to allow the affect of passing facility configuration (i.e. length and gradient) on bunching distributions to be accurately quantified as a function of traffic flow conditions.
- Passing permitted in the opposing direction at some sites to allow passing rates in such situations to be assessed.

Through use of road geometry data and right-of-way video logging acquired as part of annual high speed condition surveys of the state highway network, six passing sites having most of the above attributes were selected from a list provided by the NZ Transport Agency of all SVB's and passing lanes located on the state highway network.

The locations and characteristics of the study sites are summarised in Table 2, with photographic views and spatial maps provided in Appendix A. Site 4j in the decreasing direction was omitted from analysis due to the effect of two right turn lanes within the passing lane.

From Harwood et al (1985), the persistence of operational benefits from a passing lane, besides traffic flow conditions, appears to be highly dependent on the geometrics in the downstream area. Therefore, in this study, Rawlinson's "theoretical curve advisory speed function," which is detailed in Appendix B, was used to quantify geometric differences in downstream operational length between the study sites. This function permits 85 percentile car speeds to be calculated from horizontal curvature, cross-slope and gradient data stored in the geometry table of the NZ Transport Agency's RAMM database. However, as this function gives very high speeds on straight sections, there is a need to cap the maximum speed to the legal speed limit, which is 100 km/h for rural areas.

With reference to Table 3, two speeds have been tabulated:

1. The uncapped speed averaged over a specified distance, which provides an indication of the road alignment, the lower the speed, the more tortuous the alignment.

**Table 2: State highway passing sites selected for study**

| Category | Site ID | SH | RS | Start (m) | End (m) | RAMM Length (m) | End of Taper (from Video) (m) | End of PL Lines (from Video) (m) | Actual Length (excluding taper) from Video (m) | Direction[1] (I or D) | Location | Down-stream Road Gradient[2] | Average Gradient along PL or SVB (%) | RAMM Traffic AADT | RAMM Traffic HCV (%) | Urban /Rural | Passing in both directions |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Short PL** | **2e** | 57 | 0 | 15,800 | 15,155 | 645 | 15,812 | 15,213 | 599 | D | East Wanganui | F | 6.8 | 8,127 | 11% | R | yes |
| | **3e** | 1N | 680 | 556 | 1,113 | 557 | 561 | 1,117 | 556 | I | Central Waikato | M | 5.7 | 6,511 | 14% | R | yes |
| **Long PL** | **4j inc** | 3 | 450 | 13,314 | 14,250 | 936 | 13,305 | 14,244 | 939 | I | Newbury, East Wanganui | F | 0.4 | 7,177 | 11% | R | yes |
| | **4j dec** | 3 | 450 | 17,700 | 15,600 | n/a | n/a | n/a | 1,600[3] | D | Newbury, East Wanganui | F | -0.4 | 7,177 | 11% | R | yes |
| | **5f** | 1N | 574 | 11,879 | 10,363 | 1,516 | 11,885 | 10,488 | 1,397 | D | West Waikato | R | 0.27 | 14,368 | 11% | R | yes |
| | **6e** | 58 | 0 | 1,085 | 2,265 | 1,180 | 1,067 | 2,259 | 1,192 | I | Wellington | M | 7.2 | 13419 | 3% | R | no |
| **SVB** | **8j** | 5 | 111 | 9,900 | 9,700 | 200 | 9,886 | 9,561 | 325 | D | North of Wairakei, Central Waikato | M | 6.4 | 3,200 | 10% | R | no |

Notes: (1) D = Decreasing Direction
I = Increasing Direction

(2) F = Flat (0% ≤ | Road Gradient | < 3%)
M = Mountainous (| Road Gradient | > 6%)
R = Rolling (3% ≤ | Road Gradient | ≤ 6%)

(3) Drive-over measurement of actual length

2. The capped speed, which approximates the average free speed over the specified distance.

**Table 3: Calculated speed distributions downstream of studied passing sites**

| Study Site (Downstream Road Gradient) | Approximate Distance from Start of Merge Zone (km) | Calculated 85 Percentile Speed, with gradient effects, not capped (km/h) | Calculated 85 Percentile Speed, with gradient effects, capped to 100km/h, (km/h) |
|---|---|---|---|
| 2e (short PL) (Flat) | 0 to 0.3 | 112 | 99.98 |
| | 0.3 to 2.1 | 159 | 100.00 |
| | 2.1 to 4.6 | 142 | 99.99 |
| | 4.6 to 12.1 | 129 | 100.00 |
| 3e (short PL) (Mountainous) | 0 to 0.2 | 118 | 97.76 |
| | 0.2 to 0.5 | 122 | 100.00 |
| | 0.5 to 3.6 | 150 | 99.60 |
| | 3.6 to 9.6 | 144 | 99.35 |
| 4j (long PL) (Flat) | 0 to 0.4 | 121 | 98.70 |
| | 0.4 to 1.3 | 142 | 99.99 |
| | 1.3 to 3.7 | 146 | 99.55 |
| | 3.7 to 5.5 | 144 | 100.00 |
| 5f (long PL) (Rolling) | 0 to 0.3 | 156 | 100.00 |
| | 0.3 to 1.1 | 177 | 100.00 |
| | 1.1 to 4.0 | 160 | 99.78 |
| | 4.0 to 6.2 | 157 | 99.19 |
| 6e (long PL) (Mountainous) | 0 to 0.3 | 105 | 94.79 |
| | 0.3 to 1.7 | 125 | 96.54 |
| | 1.7 to 3.8 | 134 | 98.57 |
| | 3.8 to 7.6 | 135 | 96.62 |
| 8j (SVB) (Mountainous) | 0 to 0.3 | 159 | 100.00 |
| | 0.3 to 1.7 | 133 | 95.91 |
| | 1.7 to 3.7 | 132 | 97.30 |
| | 3.7 to 9.6 | 145 | 100.00 |

The analysis of speeds summarised in Table 3 shows sites 2e, 3e and 4j to have very similar theoretical downstream speed distributions. This similarity in theoretical downstream speed distributions suggests that direct comparisons of passing lane operational effectiveness can be made between these three sites, although sites 2e and 4j nominally cover flat road gradient and site 3e mountainous road gradient.

Site 5f has the highest uncapped speeds indicating that it has the least tortuous downstream area of the sites studied. Therefore, it is expected to display the greatest operational effectiveness for a given traffic flow as there should be more opportunity for overtaking downstream of the passing lane.

Sites 6e and 8j have almost identical speed distributions from 0.3 km after the start of the merge area and so can also be directly compared though the merge area for site 6e appears to be located on a slower alignment than site 8j.

The speed environment of the merge area, which covers the 0.3 km length of road immediately downstream of the passing facility, is shown to be similar for sites 2e, 3e, 4j and 6e. The speed environment of the merge area is markedly higher for sites 5f and 8j, and similar to one another.

# 4 Data Collection and Processing

## 4.1 Site Surveys

At each of the six study passing sites, Opus Paeroa were commissioned to carry out automated traffic surveys using Metrocount™ Plus 5600 series pneumatic tube vehicle classifiers. The surveys were structured to determine the effectiveness of the passing sites through a comparison of traffic operational conditions at three key locations: for 2 km upstream of the passing facility, within the passing facility, and for up to 12 km downstream of the passing facility.

To obtain traffic data from both treated and untreated directions, the pneumatic tubes were placed across both lanes upstream and downstream of a passing facility and "staggered" pairs within the passing facility, so one of the pair covered the "overtaking" lane of the treated direction and the lane of the untreated direction and the other of the pair covered only the "non - overtaking" lane of the treated direction. Figure 2 shows the general configuration employed, whereas the actual site configurations are given in Appendix A.

With reference to Figure 2, the location of counters 2, 3-4, 9-10 and 11 were fixed in relation to the diverge and merge areas to facilitate direct comparisons with previous work on passing lane operational effectiveness performed by Harwood et al (1985) and Koorey and Gu (2001). This required the upstream counter (counter 2) to be located 200m before the start of the diverge area, the passing facility counters (3-4) and (9-10) to be located 30m after the divergence area and 30m before the merge area respectively and the downstream counter (11) to be located 200m after the end of the merge area.

The surveys were conducted over the period 10th July 2007 to 27th July 2007, each survey period at a study passing site lasting for a minimum of 72 hours (3 full weekdays excluding weekends), so that a large proportion of high hourly flows were captured.

The vehicle classifiers were used to record traffic volumes, vehicle classes, speeds and headways (vehicle spacings). A typical vehicle classifier installation is shown in Figure 3.

## 4.2 Quality Assurance Practices

The following practices were adopted to ensure the integrity of the traffic data collected. The ratio of the logged activations between the leading tube A and the trailing tube B had to be between 95% and 105%. This is known as a sensor balance check and is routinely performed.

Additionally, the operation of selected vehicle classifiers was checked against a 15 minute video log of the traffic immediately after the classifier was installed and also before the classifier was removed to assess that it was reliably counting and classifying vehicles over the entire three day survey period.

For each study passing site, the longitudinal position of each classifier relative to a stable datum, such as RS marker post or start of passing facility, was recorded spatially (i.e. GPS northings and eastings) and linearly (trip meter). This provided assurance that the classifiers were located correctly and permits the surveys undertaken to be precisely replicated if required.

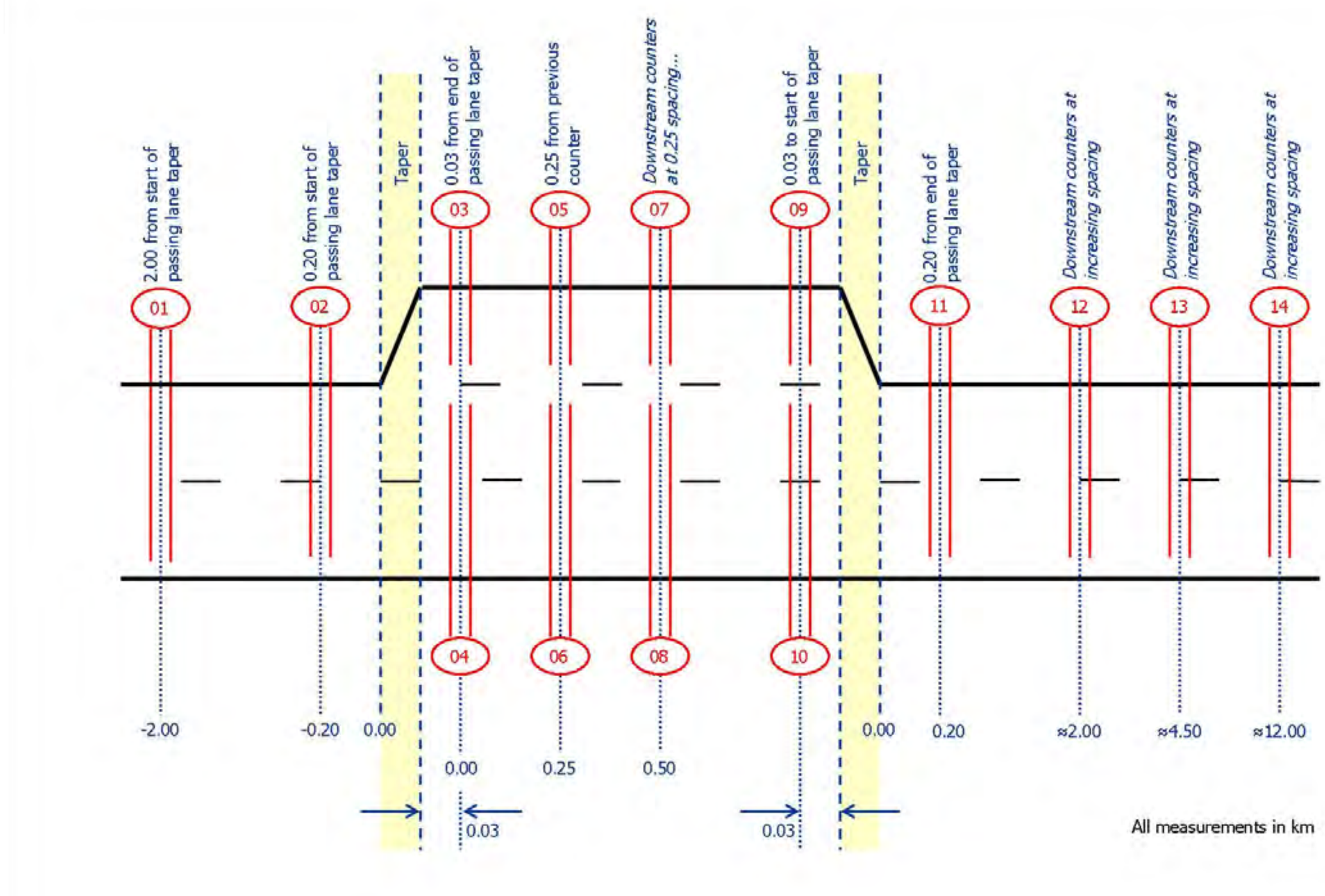


**Figure 2: Generalised vehicle classifier layout for study passing sites**

**Figure 3: Vehicle classifier installation for Site 6e, downstream location 12**

The survey field sheets that contain the vehicle classifier locational data along with the sensor balance checks are reproduced in Appendix A.

Complete data sets were obtained for all the study passing sites apart from sites 4j and 5f where surveying of traffic at some locations within the passing facility was not possible due to sensor imbalance, vandalism and placement problems.

## 4.3 Data Processing

### *4.3.1 General Processing for Each Classifier*

Data from each vehicle classifier was formatted through the MetroCount™ software to produce an "Individual Vehicles" report for each classifier. The following shows a sample of this type of report.

| DS | Axle num | Ht | YYYY-MM-DD | hh:mm:ss | Dr | Speed | Wb | Hdwy | Gap | Ax | Gp | Rho | Cl | Nm | Vehicle | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 00004db7 | 4 | 17/07/2007 | 01:15:21 | BA | 109.6 | 2.7 | 137.7 | 137 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 00004dbb | 6 | 17/07/2007 | 01:20:29 | BA | 80.1 | 6.6 | 308.2 | 308.2 | 3 | 2 | 1 | 4 | 10 | TNZ4 o | oo | | |
| 0 | 00004dc1 | 4 | 17/07/2007 | 01:24:10 | AB | 127.3 | 2.6 | 965.7 | 965.6 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 00004dc5 | 4 | 17/07/2007 | 01:26:12 | AB | 83.9 | 5.3 | 121.7 | 121.6 | 2 | 2 | 1 | 3 | 10 | TNZ3 o | o | | |
| 0 | 00004dc9 | 16 | 17/07/2007 | 01:27:54 | AB | 98 | 16.8 | 102 | 101.8 | 8 | 4 | 1 | 13 | 10 | TNZ13 o | oo | ooo | oo |
| 0 | 00004dd9 | 16 | 17/07/2007 | 01:28:47 | BA | 88.7 | 16.8 | 497.3 | 497 | 8 | 4 | 1 | 12 | 10 | TNZ12 oo | oo | oo | oo |
| 0 | 00004de9 | 4 | 17/07/2007 | 01:33:41 | AB | 107.5 | 2.9 | 347.6 | 347 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 00004ded | 6 | 17/07/2007 | 01:36:14 | AB | 96.3 | 6.9 | 152.9 | 152.8 | 3 | 2 | 1 | 4 | 10 | TNZ4 o | oo | | |
| 0 | 00004df3 | 16 | 17/07/2007 | 01:37:37 | BA | 88.1 | 17.1 | 530.8 | 530.1 | 8 | 4 | 1 | 12 | 10 | TNZ12 oo | oo | oo | oo |
| 0 | 4.00E+03 | 4 | 17/07/2007 | 01:40:12 | BA | 105.9 | 2.4 | 155 | 154.3 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 4.00E+07 | 4 | 17/07/2007 | 01:40:32 | AB | 96.9 | 2.4 | 257.8 | 257.6 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 00004e0b | 16 | 17/07/2007 | 01:44:29 | BA | 78.9 | 17.4 | 256.4 | 256.3 | 8 | 4 | 0.88 | 12 | 10 | TNZ12 oo | oo | oo | oo |
| 0 | 00004e1b | 4 | 17/07/2007 | 01:49:24 | BA | 120.3 | 2.6 | 294.7 | 293.9 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 00004e1f | 4 | 17/07/2007 | 01:50:53 | BA | 120.8 | 2.6 | 89.2 | 89.1 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 4.00E+23 | 4 | 17/07/2007 | 01:52:32 | AB | 97.8 | 3.5 | 719.7 | 719.6 | 2 | 2 | 1 | 3 | 10 | TNZ3 o | o | | |
| 0 | 4.00E+27 | 15 | 17/07/2007 | 01:52:49 | BA | 91.1 | 17 | 116 | 115.9 | 8 | 4 | 0.93 | 12 | 80010 | TNZ12 oo | oo | oo | oo |
| 0 | 4.00E+36 | 4 | 17/07/2007 | 01:53:05 | AB | 102.3 | 2.4 | 33.8 | 33.7 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 00004e3a | 4 | 17/07/2007 | 01:54:30 | AB | 102.2 | 2.7 | 84.5 | 84.4 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 00004e3e | 14 | 17/07/2007 | 01:56:33 | BA | 101.8 | 1.8 | 224.5 | 223.8 | 2 | 2 | 1 | 1 | 3010 | TNZ1 o | o | - Coerced sequence 4* | |
| 0 | 00004e3e | 14 | 17/07/2007 | 01:56:33 | AB | 101.2 | 1.8 | 0 | 0 | 2 | 2 | 1 | 1 | 3010 | TNZ1 o | o | | 3* |
| 0 | 00004e3e | 14 | 17/07/2007 | 01:56:33 | BA | 101.8 | 1.8 | 0 | 0 | 2 | 2 | 1 | 1 | 3010 | TNZ1 o | o | | 2* |
| 0 | 00004e3e | 14 | 17/07/2007 | 01:56:33 | AB | 101.2 | 1.8 | 0 | 0 | 2 | 2 | 1 | 1 | 3010 | TNZ1 o | o | | 1* |
| 0 | 00004e4c | 15 | 17/07/2007 | 01:59:52 | BA | 72.9 | 17.5 | 198.9 | 198.2 | 8 | 4 | 0.93 | 12 | 80010 | TNZ12 oo | oo | oo | oo |
| 0 | 00004e5b | 4 | 17/07/2007 | 02:01:32 | BA | 86.6 | 3.8 | 100.2 | 99.3 | 2 | 2 | 1 | 3 | 10 | TNZ3 o | o | | |
| 0 | 00004e5f | 4 | 17/07/2007 | 02:02:02 | BA | 94.2 | 2.9 | 29.3 | 29.1 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 4.00E+63 | 4 | 17/07/2007 | 02:03:02 | AB | 112 | 2.7 | 511.8 | 511.7 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 4.00E+67 | 4 | 17/07/2007 | 02:05:58 | AB | 97.1 | 2.7 | 175.9 | 175.8 | 2 | 2 | 1 | 1 | 10 | TNZ1 o | o | | |
| 0 | 00004e6b | 6 | 17/07/2007 | 02:07:05 | AB | 96.2 | 6.5 | 66.9 | 66.8 | 3 | 2 | 1 | 4 | 10 | TNZ4 o | oo | | |
| 0 | 4.00E+71 | 4 | 17/07/2007 | 02:09:52 | BA | 87.9 | 3.6 | 470.4 | 470.3 | 2 | 2 | 1 | 3 | 10 | TNZ3 o | o | | |

Highlighted in yellow is an example of a "Coerced sequence", where the activation of the vehicle classifiers does not align with expectations. It was recommended that these coerced sequences were removed and so the first step in the "cleaning" of the "Individual Vehicles" reports was to process the coerced sequences. Based on manual inspection of a number of coerced sequences, a macro was written to identify the coerced sequences, retain the first line of the sequence, and remove the other lines of the coerced sequence. (The vehicle type on the retained line was changed to "TNZ99" to denote that it had been modified.)

The "Individual Vehicles" report contains data for traffic activating the vehicle classifier in both of the possible travel directions, shown in the "Dr" column of the report by "AB" and "BA". The second step in the processing of the "Individual Vehicles" reports was to split the dataset into one dataset for the "AB" direction and another dataset for the "BA" direction. The following shows a sample of a report for the "BA" direction, as was prepared for each vehicle classifier.

| 1 | DS | Axle | Ht | YYYY-MM-DD | hh:mm:ss | Dr | Speed | Wb | Headway | Gap | Ax | Gp | Rho | Cl | Nm | Vehicle | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 41 | 0 | 00004db7 | 4 | 17/07/2007 | 01:15:21 | BA | 109.6 | 2.7 | 137.7 | 137 | 2 | 2 | 1 | 1 | 10 | TNZ1 | o | o | | |
| 42 | 0 | 00004dbb | 6 | 17/07/2007 | 01:20:29 | BA | 80.1 | 6.6 | 308.2 | 308.2 | 3 | 2 | 1 | 4 | 10 | TNZ4 | o | oo | | |
| 46 | 0 | 00004dd9 | 16 | 17/07/2007 | 01:28:47 | BA | 88.7 | 16.8 | 497.3 | 497 | 8 | 4 | 1 | 12 | 10 | TNZ12 | oo | oo | oo | oo |
| 49 | 0 | 00004df3 | 16 | 17/07/2007 | 01:37:37 | BA | 88.1 | 17.1 | 530.8 | 530.1 | 8 | 4 | 1 | 12 | 10 | TNZ12 | oo | oo | oo | oo |
| 50 | 0 | 4.00E+03 | 4 | 17/07/2007 | 01:40:12 | BA | 105.9 | 2.4 | 155 | 154.3 | 2 | 2 | 1 | 1 | 10 | TNZ1 | o | o | | |
| 52 | 0 | 00004e0b | 16 | 17/07/2007 | 01:44:29 | BA | 78.9 | 17.4 | 256.4 | 256.3 | 8 | 4 | 0.88 | 12 | 10 | TNZ12 | oo | oo | oo | oo |
| 53 | 0 | 00004e1b | 4 | 17/07/2007 | 01:49:24 | BA | 120.3 | 2.6 | 294.7 | 293.9 | 2 | 2 | 1 | 1 | 10 | TNZ1 | o | o | | |
| 54 | 0 | 00004e1f | 4 | 17/07/2007 | 01:50:53 | BA | 120.8 | 2.6 | 89.2 | 89.1 | 2 | 2 | 1 | 1 | 10 | TNZ1 | o | o | | |
| 56 | 0 | 4.00E+27 | 15 | 17/07/2007 | 01:52:49 | BA | 91.1 | 17 | 116 | 115.9 | 8 | 4 | 0.93 | 12 | 80010 | TNZ12 | oo | oo | oo | oo |
| 59 | 0 | 00004e3e | 14 | 17/07/2007 | 01:56:33 | BA | 101.8 | 1.8 | 224.5 | 223.8 | 2 | 2 | 1 | 1 | 3010 | TNZ99 | o | o | | |
| 63 | 0 | 00004e4c | 15 | 17/07/2007 | 01:59:52 | BA | 72.9 | 17.5 | 198.9 | 198.2 | 8 | 4 | 0.93 | 12 | 80010 | TNZ12 | oo | oo | oo | oo |
| 64 | 0 | 00004e5b | 4 | 17/07/2007 | 02:01:32 | BA | 86.6 | 3.8 | 100.2 | 99.3 | 2 | 2 | 1 | 3 | 10 | TNZ3 | o | o | | |
| 65 | 0 | 00004e5f | 4 | 17/07/2007 | 02:02:02 | BA | 94.2 | 2.9 | 29.3 | 29.1 | 2 | 2 | 1 | 1 | 10 | TNZ1 | o | o | | |
| 69 | 0 | 4.00E+71 | 4 | 17/07/2007 | 02:09:52 | BA | 87.9 | 3.6 | 470.4 | 470.3 | 2 | 2 | 1 | 3 | 10 | TNZ3 | o | o | | |

Highlighted in yellow is an example of a "TNZ99" where a coerced sequence was processed. The numbers in blue represent the line number from the original report. These were included at this stage as a precautionary measure to assist with tracking of the data if necessary.

### *4.3.2 Processing for a vehicle classifier on the inner lane and its adjacent vehicle classifier on the outer lane*

The physical layout of the vehicle classifiers created situations where a vehicle could activate both a vehicle classifier on the inner lane and an adjacent vehicle classifier on the outer lane. Therefore, at that point, that vehicle would be counted twice. A number of situations where this "double counting" could arise were inspected manually within development of a macro that would identify where a vehicle activated both the inner and outer lane vehicle classifiers and then the macro would remove the "second" count.

The macro identifies where the inner lane vehicle classifier and the outer lane vehicle classifier are activated almost simultaneously. These two activations could potentially truly represent one vehicle. The macro checks the speed differential between these two activations and if the differential is small then the potential remains for these two activations to actually be one vehicle. The macro compares aspects of the vehicle characteristics across the two activations and if these are closely similar the two activations are assumed as one vehicle. Based on the rule that vehicles should be travelling in the outer lane unless passing, the activation recorded in the inner lane is discarded and the activation recorded in the outer lane is retained.

The macro was developed iteratively with validations against manual inspections until an acceptably high standard of accuracy was attained.

#### *4.3.3 Processing to identify vehicles performing passing manoeuvres*

Through manual inspections of data and driving experience it was considered that some vehicles travelling in the inner lane were not performing passing manoeuvres but were simply travelling in the inner lane when they should have, perhaps more correctly, been travelling in the outer lane.

A macro was formulated to compare the data from a vehicle classifier on the inner lane and data from its adjacent vehicle classifier on the outer lane. The time of any activation on the inner lane vehicle classifier was compared with the time of the preceding outer lane vehicle classifier activation and compared with the time of the next outer lane vehicle classifier activation. At least one of these time gaps would be small if the activation in the inner lane represented a vehicle performing a passing manoeuvre. If the time gap was small, the speed recorded for the inner lane activation was compared with the speed recorded for the appropriate outer lane activation. If the speed of the inner lane activation was greater than the speed of the outer lane activation, the inner lane activation was taken as record of a true passing manoeuvre.

### 4.4 Measures of Effectiveness

The two primary measures of passing facility effectiveness used in this study were:

- percentage of following vehicles as this relates to passing lane spacing
- passing rate as this relates to the number of passes within an hour at a location along the passing lane

Percentage of following traffic is regarded as a key measure of effectiveness since it impacts on passing demand and the time spent following, two parameters frequently used to define level of service on two-lane highways. It requires each vehicle classified to be identified as free vehicle, a platoon leader, or a platoon member. For this study each vehicle with a time headway of 4-seconds or less was classified as a platoon member. The choice of the 4-seconds headway criterion to define bunching was made to allow direct comparison between the level of bunching observed at the study passing sites for different traffic volumes and that predicted using relationships proposed by Harwood et al (1985). Furthermore, 4 seconds is the shortest of the headway criteria cited in the literature, and this helps prevent classifying a vehicle as following unless this was clearly the case.

The second measure, passing rate, is defined as the number of completed passes per hour in one direction of travel. The passing rate is an appropriate measure of effectiveness because passing lanes are intended to increase the passing rate above that which would occur on a normal two-lane highway.

Two additional passing-related measures have been used in this report to complement passing rate, these being:

- ***normalised passing rate*** defined as the number of completed passes per hour per kilometre to allow direct comparisons of efficiency between passing facilities of different lengths; and
- ***percentage of passing vehicles*** which is the ratio of number of vehicles passing to the one-way flow in the treated direction expressed as a percentage, which was used to indicate changes in efficiency along a passing facility.

In deriving passing individual vehicle movements were not tracked as this is a very time consuming process. The approach used was as follows:

The proportion of flow in the inner lane is taken as:

$$\frac{(no.\ of\ \ vehicles\ in\ the\ inner\ lane)}{((no.\ of\ \ vehicles\ in\ the\ inner\ lane) + (no.\ of\ \ vehicles\ in\ the\ outer\ lane))}$$

The proportion of flow in the outer lane taken as:

*1 - (the proportion of flow in the inner lane)*

Sometimes there is a vehicle in the outer lane with no vehicle beside it (or very near it) in the inner lane, so it may be incorrect to assume that every vehicle in the outer lane is being overtaken. Each vehicle in the inner lane was therefore inspected to see if it is overtaking a vehicle in the outer lane.

The proportion of flow deemed to be passing is therefore calculated from:

$$\frac{(no.\ of\ \ vehicles\ in\ the\ inner\ lane\ with\ a\ vehicle\ beside\ it\ in\ the\ outer\ lane)}{((no.\ of\ \ vehicles\ in\ the\ inner\ lane) + (no.\ of\ \ vehicles\ in\ the\ outer\ lane))}$$

The same methodology for determining proportion of flow passing was used for the 5 passing lane sites and the 1 SVB site.

To automate the process, the macro detailed in section 4.3.3 was employed. This macro uses a ± 8 seconds time interval to establish if a vehicle can be considered “beside.” It also checks that the speed of the vehicle in the outer lane is less than the speed of the vehicle in the inner lane. Results using these two “rules” compared well against the more laborious manual inspections.

# 5 Operational Analysis Results

## 5.1 Measured Traffic Characteristics

### *5.1.1 Traffic Volume and Composition*

Directional traffic count obtained at the six study sites over the three day monitoring period was processed to obtain the average daily traffic (ADT), maximum hourly traffic volume, and percentage of the traffic that comprises light vehicles towing (LVT) and heavy commercial vehicles (HCV) i.e. TNZ class 3 and above. The results are summarised in Table 4 below.

**Table 4: Measured traffic flows for study sites**

| Category | Site ID | Location | Length of Passing Facility Excluding Tapers (m) | Down-stream Road Gradient | Average Gradient Along PL or SVB (%) | Measured Traffic Flows | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Nearest TMS 2006 AADT | 3 Day ADT (vpd) | Observed LVT & HCV as a % of 3 Day ADT (%) | Maximum One-Way Hourly Flow (vph) | Observed LVT & HCV as a % of Maximum One-Way Hourly Flow (%) |
| Short PL | 2e | SH57 RS0/15.2-15.8 | 599 | Flat | 6.8 | 8100 | 6200 | 12 | 350 (271)* | 8 - 13 |
| | 3e | SH1N RS680/0.6-1.1 | 556 | Mountain-ous | 5.7 | 6500 | 4950 | 27 | 251 (201) | 17 - 21 |
| Long PL | 4j | SH3 RS450/13.3-14.3 | 939 | Flat | 0.4 | 8600 | 9240 | 12 | 696 (349) | 5 - 10 |
| | 5f | SH1N RS574/10.4-11.9 | 1,397 | Rolling | 0.27 | 14400 | 11410 | 16 - 20 | 639 (542) | 13 - 20 |
| | 6e | SH58 RS0/1.1-2.3 | 1,192 | Mountain-ous | 7.2 | 13400 | 13600 | 8 - 10 | 871 (653) | 7 - 13 |
| SVB | 8j | SH5 RS111/9.7-9.9 | 325 | Mountain-ous | 6.4 | 3200 | 3400 | 18 | 195 (152) | 14 - 15 |

**Bracketed figures are corresponding hourly flow in opposite direction to reflect the directional split.*

With reference to Table 4, some discrepancies between the TMS 2006 traffic data from RAMM and measured 3 day ADT are observed. This suggests that caution should be exercised whenever RAMM traffic data is used to assist in site selections for traffic volume-based experimental designs. Possible explanations for the discrepancies are given in section 5.1.2 below.

Table 4 shows the directional split ranged between 0.54 and 0.67 for the six study sites. Also, the % HCV & LVT values ranged between 8-27%, with higher volume roads having generally lower values (8-12%) for % HCV & LVT. Therefore, both one-way peak hour flows and % HCV & LVT values should be used to help explain demand differences in addition to AADT values.

More detail on peak hourly flows during the week and the proportion of peak hourly flow relative to AADT is provided in Guide on Estimating AADT and Traffic Growth (Transit New Zealand, 1994). This reference also explains the traffic flow characteristics of New Zealand *rural urban fringe* and *rural strategic* state highways.

### *5.1.2 Survey 3day ADT and RAMM AADT Comparisons*

A comparison of measured ADT in Table 4 with corresponding RAMM AADT shows that the measured ADT values are generally less. The most likely reason for the discrepancy is due to RAMM AADT value being an averaged value taken over 7 full days and for *rural strategic* locations is averaged over two weeks of data (Transit NZ, 1994). Also, depending on the time of year for the survey period, the 7 day average value may be factored up to approximate a typical daily traffic flow value.

Weekends, which would typically have lower daily flows, have not been included within the averaged daily traffic flows for this study. This omission should usually increase the 3 day ADT values compared to 7 day ADT values over the same week.

This study was also undertaken during weeks 28-30 of the calendar year. Based on Transit New Zealand guidelines for estimating AADT, for these weeks, averaged daily traffic flows (ADT) in *Rural Urban Fringe* locations should be multiplied by 1.0454 - 1.1068, depending on the week (Transit, 1994). Similarly, *Rural Strategic* A and B type roads, ADTs should be multiplied by 1.0843 - 1.1639 and 1.0719 - 1.1642 respectively, depending on the survey week. Therefore, unfactored ADTs collected over the study period would probably be lower than the RAMM AADT values.

The winter school holiday, covering the period Saturday 30$^{th}$ of June to Sunday 15$^{th}$ of July can be discounted as a cause of the observed discrepancies as the automated traffic surveys took place at only one study site, site 3e, within the holiday period.

Generally, for both *Rural Urban Fringe* and *Rural Strategic* A and B sites, from Monday to Thursday, the peak daily flow is about 8 % of AADT and about 11% of AADT for Friday. Therefore, 3 day ADTs that include Friday data would be higher than 3 day ADTs that do not include Friday.

### *5.1.3 Peak Hour Flow Characteristics*

Table 5 compares peak hour flows in terms of their directional split and proportion of 3 day ADT.

**Table 5: Comparison of peak hour flow characteristics for each site**

| Site ID | 3 Day ADT | Peak Flow (both directions) (vph) | Peak Flow (one-way) (vph) | Directional Split (%) | Peak Hour as Proportion of Surveyed ADT (%) |
|---|---|---|---|---|---|
| 2e | 6,200 | 621 | 350 | 56/44 | 10.0 |
| 3e | 4,950 | 452 | 251 | 56/44 | 9.1 |
| 4j | 9,240 | 1,045 | 696 | 67/33 | 11.3 |
| 5f | 11,410 | 1,181 | 639 | 54/46 | 10.4 |
| 6e | 13,600 | 1,524 | 871 | 57/43 | 11.2 |
| 8j | 3,400 | 347 | 195 | 56/44 | 10.2 |

For sites 2e, 3e, 5f and 8j, a reasonable approximation for the directional split is 55/45 and the peak hourly flow as a proportion of the surveyed ADT is about 10% peak. These sites are on rural strategic non-recreational routes.

Sites 4j is on the urban fringes of Palmerston North. Site 6e is on the urban fringes of Lower Hutt. These sites also experience rural commuter activity. Therefore, the peak hourly flow as a proportion ADT and the directional flow would be stronger than for the other study sites.

For site 4j, a 65/35 directional split seems appropriate. Site 6e is on a rural urban fringe route but lies between two large urban areas, namely Porirua and Lower Hutt, which could explain why its directional split is close to 55/45. For both sites 4j and 6e, peak hourly flow is assumed to be 11% of ADT rather than the 10% assumed for sites 2e, 3e, 5f and 8j.

### *5.1.4 Relationship of Study Peak Hour Flows to AADT Ranges in Policy Framework*

In applying the NZ Transport Agency's long-term framework for passing and overtaking treatments, design flows are used. For rural strategic non-recreational routes, it is suggested that design peak hourly flows are taken as 10.5% of AADT and 55%/45% directional split. The 10.5% of AADT value approximates to the 125$^{th}$ percentile hour or with reference to Table A7.2 of the Economic Evaluation Manual (Land Transport New Zealand, 2007) about 95% of all hourly flows recorded in a year will be at or below this value. If 8% - 9% of AADT was used to estimate the peak hourly flow near the end of the projects design life, the design peak hourly flow would be exceeded about 37% of the time rather 5% of the time. Therefore, adoption of the 125$^{th}$ percentile hour value will result in a longer service life for the passing facility.

For urban fringe routes, it is suggested that peak flows are taken as 12% of AADT and 65%/35% directional split. The value of 12% has been derived by relative scaling of average observed values for both rural strategic (9.9%) and urban fringe route (11.25%) from Table 5 and the design value of 10.5% for rural strategic derived from the Economic Evaluation Manual.

The resulting equivalencies between projected AADT and peak hourly flow have been tabulated in Table 6 for both rural strategic non-recreational and rural urban fringe routes. As one-way peak flows in the treated direction relates to passing length, the one-way peak flows have been kept the same for both routes.

Using the characteristics identified in Table 5 for different types of route, maximum hourly flows have been converted to AADT ranges. Therefore, the study's traffic count data can be related to the projected AADTs in Table 1.

From Table 6, study sites on rural urban fringe routes will have lower opposing flows for the same maximum peak hour flow. On rural urban fringe routes with good overtaking visibility, the lower amount of opposing traffic could affect the rate of increase in bunching and hence the spacing needed between passing facilities. This effect would reduce as opposing volumes increase. Therefore, for the same maximum peak one-way flow, spacings derived from the survey results for rural urban routes with good overtaking opportunities and marked directional split (i.e. 65%/35%) are expected to be greater than for rural non-recreational routes with comparable overtaking sight distance.

**Table 6: Design ADT as one-way hourly flow equivalencies**

| Rural Strategic Non-Recreational Routes | | Rural Urban Fringe Routes | |
|---|---|---|---|
| **Design AADT (vpd)** | **Peak Hour Flow[1] (vph)** | **Design AADT[2] (vpd)** | **Peak Hour Flow[1] (vph)** |
| 2,000 | 120 (90)[3] | 1,500 | 120 (60) |
| 4,000 | 230 (190) | 2,900 | 230 (120) |
| 5,000 | 290 (240) | 3,800 | 290 (160) |
| 7,000 | 400 (330) | 5,200 | 400 (220) |
| 10,000 | 580 (470) | 7,400 | 580 (310) |
| 12,000 | 690 (570) | 8,800 | 690 (370) |
| 20,000 | 1160 (950) | 14,800 | 1160 (620) |
| 25,000 | 1440 (1180) | 18,500 | 1440 (780) |

**Notes:**
1. Peak hour flows rounded to the nearest 10.
2. Projected AADT for rural urban fringe rounded to nearest 100.
3. Flow in opposing (untreated) direction shown in brackets.

### *5.1.5 Variation of Traffic Volume Downstream of Study Site*

With reference to Appendix A, it can be seen that side roads were present downstream of the passing facility for all the study sites. In most cases they were of a minor nature and so did not appreciably affect traffic flows, apart from the following five situations:

- Site 2e, between counters 11 and 12 where southbound traffic leaves SH57 to join SH1N (approximately 40% reduction in traffic flow).
- Site 4j, between counters 15 and 16 where traffic from Kairanga Bunnythorpe Road (SH54) joins SH3 (approximately 27% increase in traffic flow).
- Site 5f, between counters 18 and 19 where traffic from Karapiro Road and a lesser extent Gorton and Tunakawa Roads join SH1N (approximately 16% increase in traffic flow).
- Site 5f, between counters 19 and 20 where northbound traffic turns off SH1N into Hydro Road and Hickey Road (approximately 7% decrease in traffic flow).
- Site 8j, between counters 7 and 8 where eastbound traffic turns off SH5 into Palmer Mill Road (approximately 10% decrease in traffic flow).

Therefore, the ability to investigate the effectiveness of the passing facility over a significant distance downstream was curtailed for four of the six study sites, with sites 5f and 8j being particularly affected.

## 5.2 Measured Platooning Characteristics

### *5.2.1 Upstream Bunching Levels*

Bunching reflects the combined influence of traffic volume, vehicle composition and upstream geometrics (along with other factors) on the traffic entering the passing facility. Therefore, the relationship between upstream bunching and directional hourly traffic volume was investigated for each of the six study sites in order to identify similarities and differences between the sites.

With reference to Table 7, a simple linear relationship applies to all the study sites. This model is sufficient to explain between 95% and 99% of the variation in the dependent variable (% following) for the range of one-way hourly volumes covered.

Table 7 shows that the study sites fall into two distinct groups, those where the upstream % following equals approximately 0.13 of the one-way hourly flow (sites 4j, 5f and 6e i.e. the longer passing facilities) and where the upstream % following equals approximately 0.2 of the one-way hourly flow (sites 2e, 3e, and 8j i.e. shorter passing facilities and the SVB). It is also noted that the slopes of 0.13 and 0.20 are entirely consistent with those previously reported by Koorey and Gu (2001) for the Otaihanga passing lane and Kaimai SVB sites respectively, thereby providing a degree of confidence that the selected study sites are representative of the state highway network.

**Table 7: Study site upstream bunching – directional hourly flow relationships**

| Category | Site ID | Location | Downstream Road Gradient | Upstream Bunching Relationship % Following = A x Flow (vph) | | One-way Hourly Flow Range Investigated (vph) |
|---|---|---|---|---|---|---|
| | | | | A | $r^2$ | |
| Short PL | 2e | SH57 RS0/15.2-15.8 | Flat | 0.203 | 0.982 | 10 - 260 |
| | 3e | SH1N RS680/0.6-1.1 | Mountainous | 0.208 | 0.988 | 10 - 240 |
| Long PL | 4j | SH3 RS450/13.3-14.3 | Flat | 0.145 | 0.971 | 21 - 475 |
| | 5f | SH1N RS574/10.4-11.9 | Rolling | 0.131 | 0.982 | 40 - 520 |
| | 6e | SH58 RS0/1.1-2.3 | Mountainous | 0.120 | 0.953 | 20 - 688 |
| SVB | 8j | SH5 RS111/9.7-9.9 | Mountainous | 0.211 | 0.986 | 10 - 190 |

### *5.2.2 Upstream Headway Distributions*

Time headway is the interval between individual vehicles measured from head to head as they pass a given point (Pignataro, 1973) The relationship between vehicle spacing and headway is dependent on speed, with:

$$Headway\ (s) = \frac{Spacing\ (m)}{Speed\ (m/s)} \quad \ldots(5.1)$$

A comparison of the time headway distribution immediately upstream of the passing facility, averaged over the three day monitoring period, is given in Figure 4. As previously mentioned, each vehicle with a time headway of 4 seconds or less is classified in this study as being a platoon member. With reference to Figure 4, 24% to 53% of the directional traffic flow had a time headway of 4 seconds or less, the proportion increasing with increasing 3 day ADT.

To drive safely behind the vehicle in front in a steady stream of traffic, motor vehicle drivers are advised to keep two second headway (http://www.landtransport.govt.nz/roadcode/). This headway is sufficient for the vast majority of drivers to prevent a rear-end collision with the vehicle in front, particularly where the traffic situation is not very complex. Table 8 shows the proportion of the flow immediately upstream of the passing facility with a headway of 2 seconds or less, corresponding to the situation where the following distance is at or less than the recommended safe following distance, relative to the proportion of the flow immediately upstream of the passing facility with a headway 4 seconds or less (i.e. traffic considered to be following).

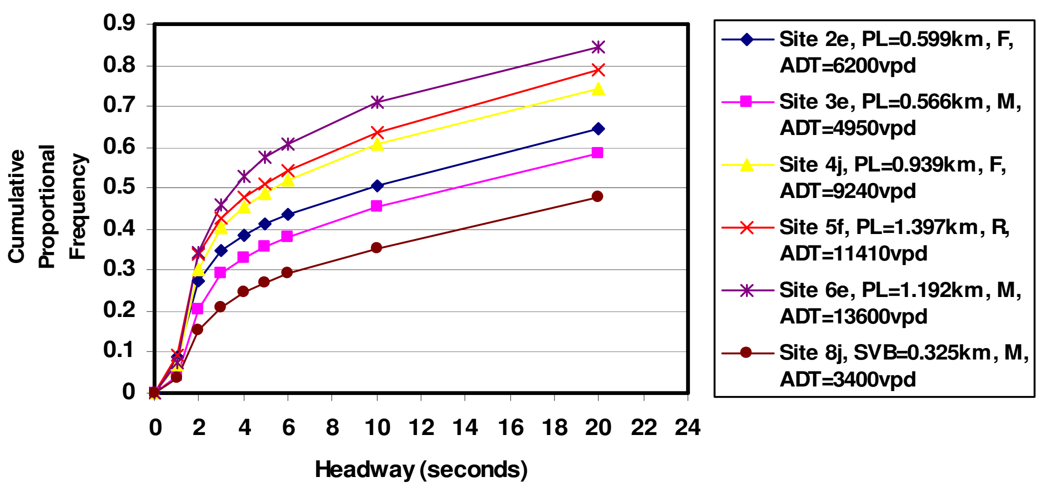


**Figure 4: Distribution of headways immediately upstream of passing facility**

With reference to Table 8, study sites 2e, 4j and 5f have a slightly higher proportion of bunched flow with a headway of 2 seconds or less, suggesting that passing rates immediately after the diverge area at these 3 study sites could be marginally higher than for the other study sites as passing is more likely since the passing vehicle has less catching up to do in order to execute the passing manoeuvre.

**Table 8: Proportion of bunched flow with headway less than or equal to 2 seconds**

| Site ID | Headway Characteristics Upstream of Passing Facility | | |
|---|---|---|---|
| | Proportion of Directional Flow with Headway ≤2 s | Proportion of Directional Flow with Headway ≤4 s | Headway Ratio ≤2 s : ≤ 4 s |
| 2e | 0.27 | 0.39 | 0.69 |
| 3e | 0.20 | 0.33 | 0.61 |
| 4j | 0.30 | 0.45 | 0.67 |
| 5f | 0.34 | 0.48 | 0.71 |
| 6e | 0.34 | 0.53 | 0.64 |
| 8j | 0.15 | 0.24 | 0.63 |

### *5.2.3 Bunching Distributions Upstream and Downstream of Studied Sites*

Figure 5 gives a conceptual illustration of the effect of a passing lane on traffic operations on a two-lane highway reproduced from Harwood et al, 1988. The solid line in Figure 5 shows fluctuations in the spot percentage of following vehicles (% following) on a normal two-lane highway brought about by available overtaking sight distance. Introduction of the passing lane produces a significant decrease in the % following within the passing lane, which then stabilizes. Downstream of the passing lane, the % following increases gradually until it reaches that for the normal two-lane highway.

Figure 5 also shows the concept of effective length, which is the length of passing lane plus the distance downstream to the point where % following matches the level immediately before entering the passing lane.

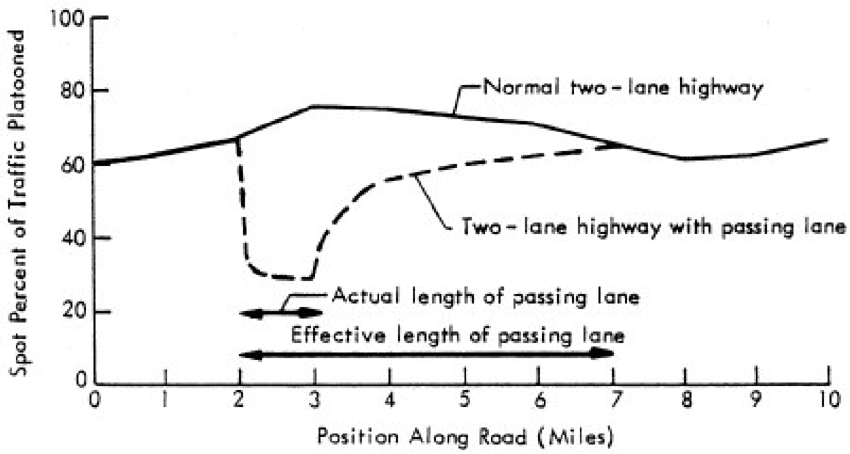


**Figure 5: Illustrative example of the effect of a passing lane on two-lane highway traffic operations (reproduced from Harwood et al, 1988)**

The variation in upstream and downstream spot % following for the six study sites at their maximum one-way hourly flows are compared for 2 and 4 seconds headway in Figures 6 and 7 respectively. The start of the passing facility is at distance 0 m.

Generally, the observed % following distributions shown in Figures 6 and 7 match the conceptual distribution of Figure 5, though the increase in % following downstream of the passing facility appears to be more linear than logarithmic as shown in Figure 5.

The between site differences in % following distributions are attributed to the proportion of non-cars as well as road geometry and traffic flow characteristics. The rate at which the % following increases downstream of the passing facility was found to vary between 0.4% and 1.6% per kilometre for 2-seconds headway criterion and 0.3% and 1.4% for 4-seconds headway criterion.

Plots of the spatial variation in bunching based on 4-seconds headway criterion upstream and downstream of the passing facility for different ranges of hourly directional flow are additionally presented in Appendix A for each of the six study sites. Within site differences seen in the bunching distributions are attributed primarily to differences in the proportion of heavy commercial

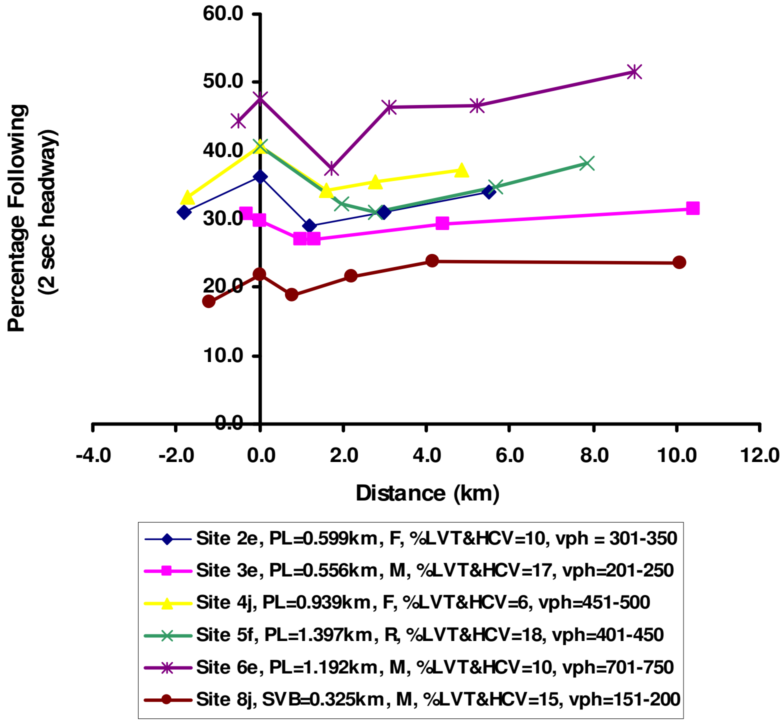


**Figure 6: Percent following distribution based on 2-seconds headway criterion**

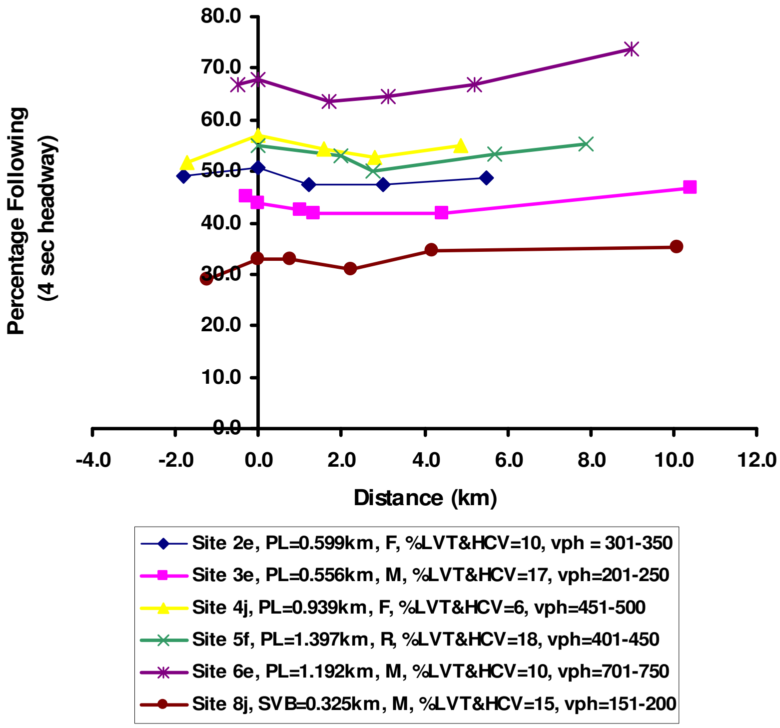


**Figure 7: Percent following distribution based on 4-seconds headway criterion**

vehicles (HCV), recreational vehicles (RV) and light vehicles towing (LVT) among the traffic flow ranges investigated

An important parameter in evaluating the operational effectiveness of a passing facility is the difference in percentage of following vehicles, immediately upstream and downstream. Table 9 shows the effect of the studied sites on % following. Immediately downstream of the passing facility, the average reduction in % following based on a 4-seconds headway criterion is 4.4 percent. This is comparable to the 5.9 percent measured by Harwood et al. (1985) over similar average traffic flows (35 to 560 vehicles per hour) at 12 passing-lane and 3 short four-lane sites.

With reference to Table 9, it can be seen that for each site the difference in % following immediately upstream and downstream is least for the highest traffic flow interval recorded. While this may indicate that the passing facility is approaching capacity, it could also be due to very little data being obtained at high traffic flows. Therefore, not much significance can be attached to this observation.

**Table 9: Effect of Passing Lane on Percentage of Following Vehicles**

| Site | Flow Rate (vph) | No. of Hourly Readings | LVT& HCV (%) | Percentage of Following Vehicles (4 sec headway criterion) | |
|---|---|---|---|---|---|
| | | | | Immediately Upstream | Upstream - Downstream Reduction |
| 2e | 301- 350 | 4 | 10 | 50.6 | 3.2 |
| | 201 - 300 | 17 | 12 | 45.0 | 5.7 |
| | 101 - 200 | 15 | 11 | 32.3 | 5.5 |
| 3e | 201 - 250 | 8 | 17 | 43.9 | 2.2 |
| | 101 - 200 | 25 | 20 | 34.1 | 2.8 |
| 4j | 451 - 500 | 6 | 6 | 56.9 | 4.2 |
| | 301 - 400 | 15 | 11 | 51.1 | 5.8 |
| | 201 - 300 | 13 | 13 | 45.8 | 5.8 |
| 5f | 401 - 450 | 11 | 18 | 54.9 | 4.6 |
| | 301 - 400 | 20 | 17 | 50.3 | 5.7 |
| 6e | 701 - 750 | 5 | 10 | 68.0 | 4.4 |
| | 651 - 700 | 5 | 13 | 66.3 | 4.9 |
| | 301 - 400 | 14 | 13 | 45.0 | 7.9 |
| 8j | 151 - 200 | 5 | 15 | 33.0 | 1.9 |
| | 101 - 150 | 21 | 19 | 27.0 | 5.0 |

## 5.3 Passing Lane Spacing

Passing lane spacing is the distance from the end of one auxiliary lane to the start of the next in the same direction. Providing some guidance on passing lane spacing is helpful prior to establishing potential locations. It is also an indication of how practical it is to achieve desired levels of service.

Harwood and Hoban (1987) suggested that the desired passing lane frequency, which is the distance from the start of one passing lane to the start of the next downstream passing lane in the same direction of travel, should be equal to the effective length of the preceding passing lane. Therefore, the desired passing lane spacing is the distance from the end of the passing lane to the point downstream where traffic conditions return to the bunching level at the beginning of the passing lane. For the purposes of this report, this distance is referred to as the downstream operational length of the passing lane.

The following methodology was adopted to derive downstream operational lengths from the bunching distributions obtained at the six sites studied to enable comparisons with local and overseas design guidelines for passing lane spacing.

For each measurement location upstream and downstream of the passing facility, percentages of traffic bunched were calculated over hourly intervals for headways of 2 seconds and 4 seconds. The 2-seconds headway criterion was selected because a study by Gallis et al (1997) concluded that, for speeds greater than 80 km/h, headways more than 3 seconds did not encourage drivers to perform passing manoeuvres. In other words, for rural roads, drivers are more likely to feel their trip is being impeded and are more inclined to pass if the headways are less than 3 seconds. As previously mentioned, a 4-seconds headway criterion was also selected to enable direct comparisons with the study by Harwood et al (1985).

A linear regression was performed on hourly values of percentage of following traffic measured at two or more locations downstream of the merge area of the passing lane. In choosing the data to be regressed, care was taken to eliminate values of percentage of following traffic that may have been affected by traffic flows generated by side roads and/or bunching caused by traffic merging at the end of the passing lane.

The values regressed were the distance from the end of the passing lane to the downstream location (in metres), the "y" parameter, and the percentage of following traffic at this location, the "x" parameter. Therefore, the downstream operational length could be readily determined by inputting into the resulting regression equation the percentage of following traffic immediately upstream of the passing lane.

The "FORECAST" function in Microsoft$^{TM}$ Excel enabled this regression modelling process to be automated so that estimates of downstream operational length could be calculated for every hour of data collected over the three day monitoring period. This allowed the influence of traffic volume, traffic composition, and time of day on downstream operational length to be investigated.

The resulting hourly estimates of downstream operational length with similar traffic flow and composition were combined for each site to obtain the average values summarised in Table 10. In calculating the average hourly estimates, only hourly records that showed increasing % following vehicles with increasing distance downstream of the merge area were utilised so that the conceptual model shown in Figure 5 was conformed to.

With reference to Table 10, it can be seen that there is considerable variation in the downstream operational length calculated, within and between sites, with downstream operational lengths spanning 1km to 22 km. This supports previous research (Harwood et al 1988) that suggests downstream operational length is dependent on passing lane length, traffic flow and composition and downstream passing opportunities. The entry "PL ineffectual" in Table 10 signifies that the

**Table 10: Estimates of Downstream Operational Length**

<table>
<tr><th rowspan="2">Site ID (Length)</th><th rowspan="2">Downstream Road Gradient (%)</th><th rowspan="2">Average Gradient Along PL or SVB (%)</th><th rowspan="2">Observed ADT (vpd)</th><th rowspan="2">Observed Directional Flow (vph)</th><th rowspan="2">Observed LVT & HCV (%)</th><th rowspan="2">Headway Criterion (sec)</th><th colspan="2">Estimated Downstream Operational Length (km)</th></tr>
<tr><th>Range</th><th>Average</th></tr>
<tr><td rowspan="4">2e (599m)</td><td rowspan="4">Flat (0%-3%)</td><td rowspan="4">6.8</td><td rowspan="4">6200</td><td>≥ 260</td><td>8 - 13</td><td>4</td><td colspan="2">PL ineffectual</td></tr>
<tr><td>261 -328</td><td>8 - 13</td><td>2</td><td>4.1 - 11</td><td>6.8</td></tr>
<tr><td rowspan="2">119 - 250</td><td>12 - 17</td><td>4</td><td>1.9 – 5.4</td><td>3.4</td></tr>
<tr><td>9 - 14</td><td>2</td><td>2.1 – 10.5</td><td>5.3</td></tr>
<tr><td rowspan="4">3e (556m)</td><td rowspan="4">Mountainous (>6%)</td><td rowspan="4">5.7</td><td rowspan="4">4950</td><td>≥ 190</td><td>17 - 21</td><td>4</td><td colspan="2">PL ineffectual</td></tr>
<tr><td>190 - 250</td><td>17 - 21</td><td>2</td><td>6.4 – 13.7</td><td>10.0</td></tr>
<tr><td rowspan="2">117 - 188</td><td>11 - 19</td><td>4</td><td>3.5 – 8.6</td><td>4.8</td></tr>
<tr><td>13 - 19</td><td>2</td><td>2.3 – 18.9</td><td>6.3</td></tr>
<tr><td rowspan="2">4j (939m)</td><td rowspan="2">Flat (0%-3%)</td><td rowspan="2">0.4</td><td rowspan="2">9240</td><td rowspan="2">343 - 487</td><td rowspan="2">5 - 10</td><td>4</td><td>3.8 – 11.1</td><td>6.7</td></tr>
<tr><td>2</td><td>3.4 – 21.7</td><td>13.5</td></tr>
<tr><td rowspan="2">5f (1,397m)</td><td rowspan="2">Rolling (3%-6%)</td><td rowspan="2">0.27</td><td rowspan="2">11410</td><td rowspan="2">355 - 558</td><td rowspan="2">13 - 20</td><td>4</td><td>2.7 – 9.7</td><td>4.8</td></tr>
<tr><td>2</td><td>3.4 – 12.8</td><td>6.9</td></tr>
<tr><td rowspan="4">6e (1,192 m)</td><td rowspan="4">Mountainous (>6%)</td><td rowspan="4">7.2</td><td rowspan="4">13600</td><td>≥688</td><td>7 - 13</td><td>4</td><td colspan="2">PL ineffectual</td></tr>
<tr><td>693 - 805</td><td>7 - 13</td><td>2</td><td>1.3 – 5.4</td><td>3.9</td></tr>
<tr><td rowspan="2">530 - 680</td><td rowspan="2">11 - 13</td><td>4</td><td>1.8 – 6.3</td><td>3.0</td></tr>
<tr><td>2</td><td>1.6 – 5.9</td><td>3.2</td></tr>
<tr><td rowspan="4">8j (325m)</td><td rowspan="4">Mountainous (>6%)</td><td rowspan="4">6.4</td><td rowspan="4">3400</td><td>≥ 148</td><td>12 - 19</td><td>4</td><td colspan="2">PL ineffectual</td></tr>
<tr><td>170 - 192</td><td>14 - 15</td><td>2</td><td>1.2 – 3.6</td><td>2.4</td></tr>
<tr><td rowspan="2">114 - 143</td><td>12 - 22</td><td>4</td><td>1.0 – 11.7</td><td>4.8</td></tr>
<tr><td>12 -18</td><td>2</td><td>3.5 – 14.5</td><td>8.4</td></tr>
</table>

% following vehicles is relatively constant across the measurement locations upstream and downstream of the passing lane, indicating that the passing lane has minimal impact on traffic operations at the specified directional flow.

However, the following general trends emerge:

- For 4-seconds headway, the downstream operational length of passing lanes decreases with increasing traffic volume.
- For 2-seconds headway, the average downstream operational length of passing lanes increases with increasing traffic volume. However, the upper bound of the range of downstream operational lengths is usually higher or similar to that for lower traffic volumes.
- The downstream effectiveness of a passing lane declines as the headway increases. Typically, for the same hourly traffic flow, the downstream operational length calculated using 2-seconds headway is between 1.1 and 2 times that calculated using 4-seconds headway.
- For 4-seconds headway, some passing lanes provide no improvement in traffic operations once a critical volume of traffic is reached.

To investigate the reasonableness of the calculated downstream operational lengths in Table 10 as target passing lane spacings, the tabulated average values were compared with guideline passing lane spacings provided in Technical Bulletin DS 98003 of the Ministry of Transportation and Highways, British Columbia. These guideline spacings were selected for comparison because they represent the minimum spacing between passing lanes and are based on the time it takes for platoons to re-form.

With reference to Table 11, there is good agreement between the New Zealand (2-seconds headway based) and British Columbia passing lane spacings, suggesting that traffic composition and terrain may not be as important as traffic volume for setting passing lane spacings. This was further investigated by regressing traffic related parameters directional flow (vph) and % LVT& HCV, geometry related parameters gradient and theoretical uncapped 85 percentile speed (from Table 3), and passing lane length against hourly derived downstream operational length to establish the degrees of correlation. The regression analysis indicated that for the sites studied, downstream operational length is most correlated to gradient along the passing lane/SVB, followed by theoretical uncapped 85 percentile speed, downstream gradient and directional flow but for all cases the correlation can be regarded as being weak ($r^2 \leq 11\%$). There was no correlation with passing lane length and % LVT & HCV.

On the basis of this finding, guidelines for passing lane and SVB spacings should consider downstream road gradient and sight distances/downstream passing opportunities in addition to traffic volume.

**Table 11: Passing lane and slow vehicle bay spacing as a function of AADT**

| **AADT (vpd)** | **Spacing between passing lanes (km)** | |
|---|---|---|
| | British Columbia guideline minimums | Inferred from NZ measurements ( 2 sec headway downstream operational length for similar flow ranges) |
| 1001 - 3000 | 9.6 | 8.4 average, 3.5-14.5 (site 8j, rural strategic) |
| 3001 - 5000 | 8.0 | 10 average, 6.4-13.7 (site 3e, rural strategic) |
| 5001 - 7000 | 6.4 | 6.8 average, 4.1-11 (site 2e, rural strategic) |
| 7001 - 9000 | 4.4 | 13.5 average*, 3.4-21.7 (site 4j, rural urban fringe)<br>6.9 average, 3.4-12.8 (site 5f, rural strategic) |
| >9000 | 4.0 | 3.9 average, 1.3-5.4 (site 6e, rural urban fringe) |

* 7.6 km if calculated using regression model given by equation 6.12

## 5.4 Passing Behaviour Within Passing Facility

### *5.4.1 Passing Rates*

Because individual vehicles were identified during the data reduction, the number of passing vehicles and the number of vehicles that were passed could be determined using the methodology described in section 4.4. Plots of completed passes per hour at all measurement locations within the passing facility of each of the study sites are presented in Figures 8 to 13.

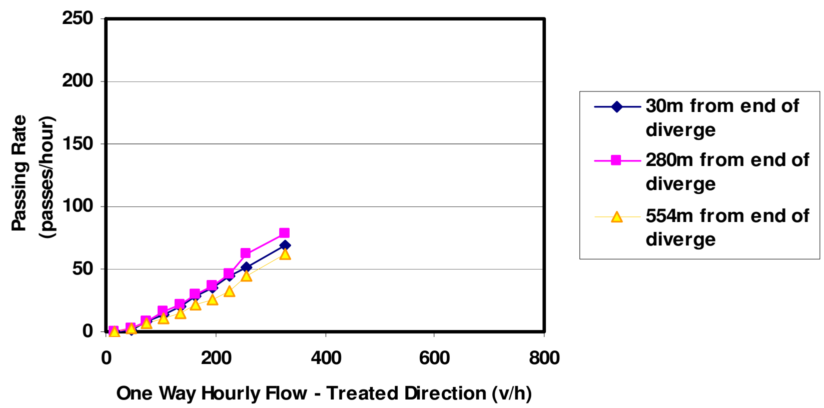


**Figure 8: Passing Rate Distribution – Site 2e, PL = 0.599 km, PL Grade = 6.8%**

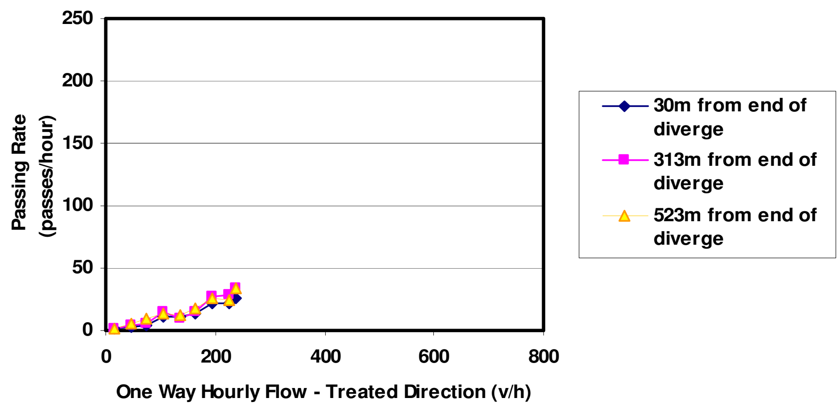


**Figure 9: Passing Rate Distribution – Site 3e, PL = 0.556 km, PL Grade = 5.7%**

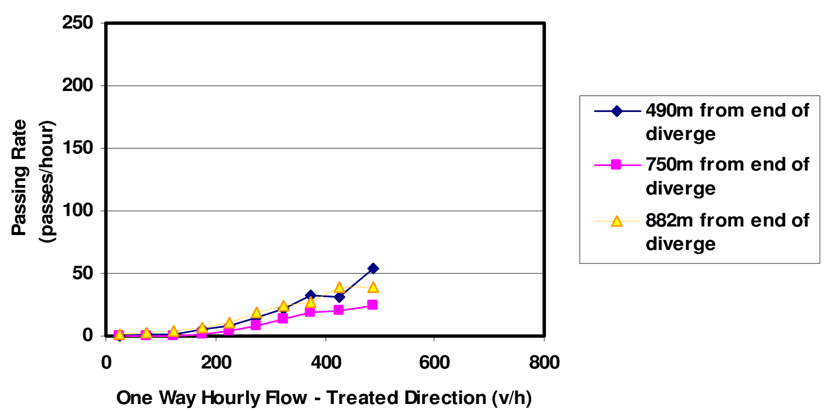


**Figure 10: Passing Rate Distribution – Site 4j, PL = 0.939 km, PL Grade = 0.4%**

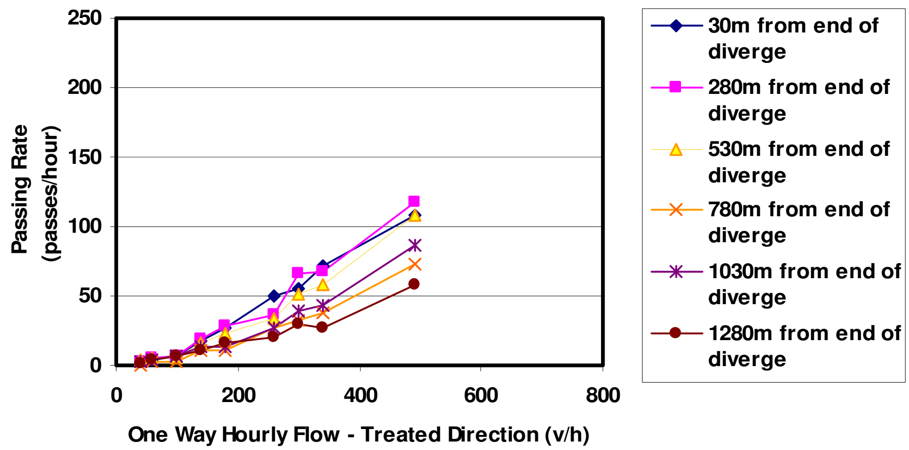


**Figure 11: Passing Rate Distribution – Site 5f, PL = 1.397 km, PL Grade = 0.3%**

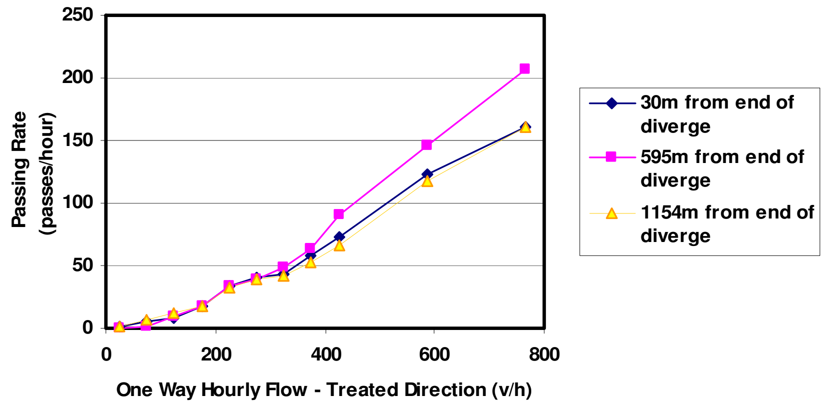


**Figure 12: Passing Rate Distribution – Site 6e, PL = 1.192 km, PL Grade = 7.2%**

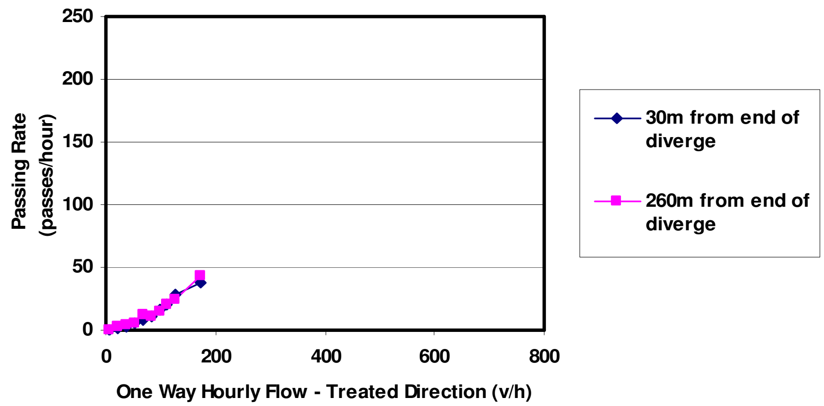


**Figure 13: Passing Rate Distribution - Site 8j, SVB = 0.325 km, PL Grade = 6.4%**

Apart from site 4j, passing rates were available near the start, middle and end of the passing facility so any differences in passing rate over the length of the passing facility could be highlighted. Equipment failure at site 4j resulted in passing rate data being available for only the second half of the passing facility.

Noteworthy characteristics of the passing rate plots are as follows:

- The passing rate increases with increasing flow.
- Up to 200 - 300 vehicles per hour (vph), the passing rate is fairly constant throughout the length of the passing lane.
- Above 200 – 300 vph, for short passing lanes the highest passing rates occur near the middle of the passing lane whereas for long passing lanes the highest passing rates occur a quarter of the way down the passing facility.
- There are no clear trends with downstream road gradient or passing lane length.

### *5.4.2* Inter-Site Comparisons

To identify any trends with terrain or passing lane length, the six studied passing sites were compared on the basis of passing rate, normalised passing rate and percentage of passing vehicles. Definitions of these three passing related parameters are provided in section 4.4.

The comparisons are based on passing manoeuvres observed at the middle of the passing facility and that these are representative of the facility as a whole.

Figure 14 shows the SVB and short passing lanes (sites 2e, 3e and 8j) to have higher passing rates for a given hourly flow than the long passing lanes (sites 4j, 5f and 6e). This result is most likely due to the SVB and short passing lanes having higher percentage of vehicles bunched upstream of the facility (refer Table 7, section 5.1.3). Also, the degree to which passing rate increases with traffic flow is similar across all sites. The highest passing rate observed was 207 passes per hour and occurred on the second longest passing facility, site 6e.

It was expected that passing facilities with mountainous road gradient would have higher passing rates compared to those with rolling road gradient. This was not observed, possibly because, of the two mountainous sites, site 3e was not as tightly bunched leading up to the passing facility because of a 1.4 km straight with good visibility ending about 1.5 km upstream of the site and another 300 m of clear sight distance before the passing lane diverge and for site 6e uneven bunching due to traffic lights. It will also be noted that for the long passing facilities, site 4j (flat road gradient) is under performing at higher flows (> 100 vph) compared to sites 5f (rolling) and 6e (mountainous). However, site 4j's under-performance is probably because it is regularly targeted by NZ Police for mobile speed enforcement.

As passing facilities reach their limit, they will not be able to accommodate any more passes within their length and will start to record a constant or possibly a drop off in the percentage of passing vehicles. With reference to Figure 15, it will be seen that this "saturation" point is reached for site 8j (SVB site) and short passing lane site with flat road gradient (site 2e) for one-way hourly traffic flows of about 130 and 260, respectively.

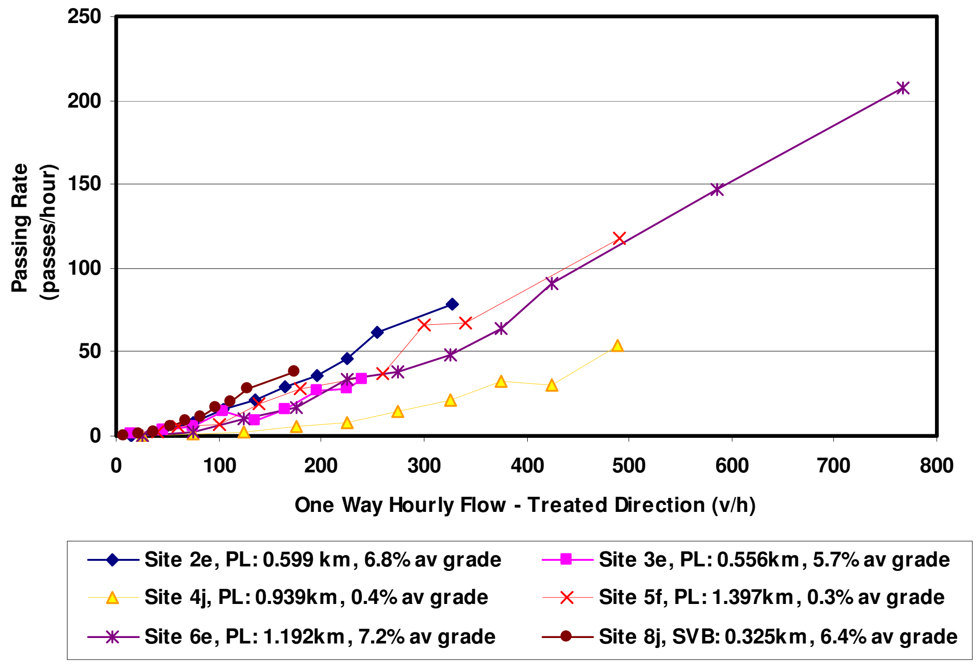


**Figure 14: Comparison of passing rate distributions for study sites – counter at middle of passing facility**

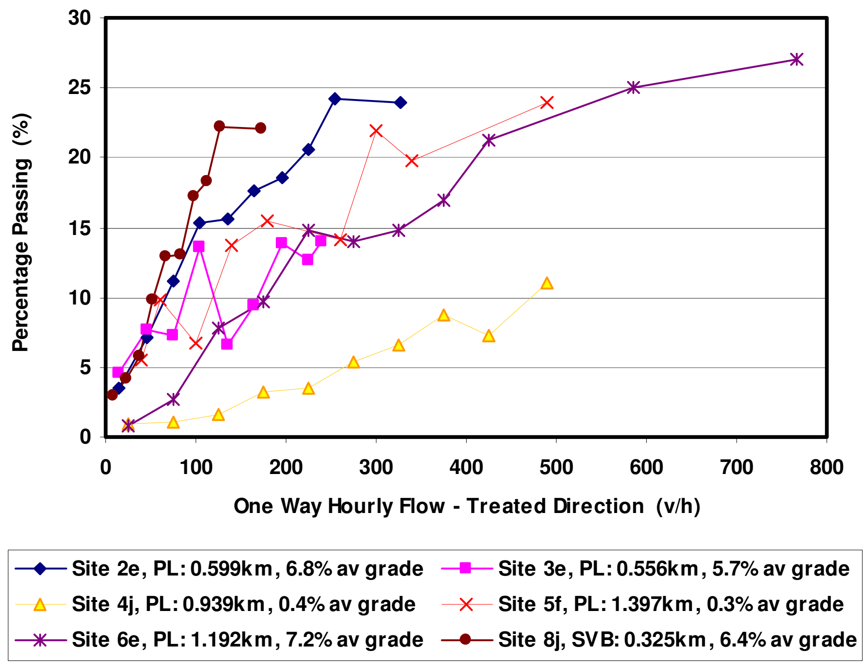


**Figure 15: Percent passing distributions for study sites – counter at middle of passing facility**

There is also an indication that the short passing lane with mountainous road gradient (site 3e) is less efficient than the short passing lane with flat road gradient (site 2e) up to 240 vph, due to good overtaking opportunities in the 3 km leading up to its diverge, although site 3e does follow trends for the longer passing treatments with rolling and mountainous road gradient (sites 5f and 6e) at similar one-way hourly traffic flows.

Figure 16 shows at lower flows (i.e. up to about 200 vph one-way) that per kilometre of facility, the most effective with respect to passing rates are SVB's, followed by short passing lanes, with long passing lanes being the least effective. No significant differences in normalised passing rate were observed between rolling and mountainous road gradient when comparing both short and long passing lanes located on each of these road gradients.

The passing facility located on the flat, (site 4j), was shown to be the least effective per kilometre length, probably due to there being a highly visible NZ Police presence and/or it may be harder to pass on the flat where operating speed differentials would be smaller than with rolling and mountainous road gradient. The most likely explanation is that site 4j is targeted by NZ Police for mobile speed enforcement as site 5f was also located on a relatively flat gradient but performed adequately although site 5f had a slightly higher percentage of vehicles bunched upstream than site 4j (refer Table 8, section 5.2.2).

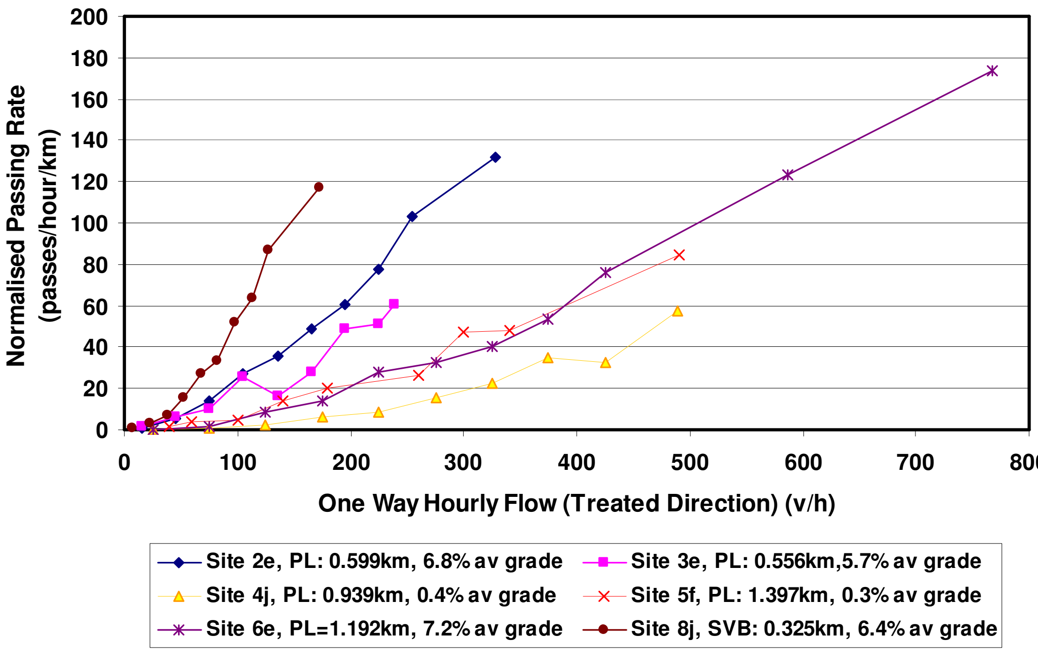


**Figure 16: Normalised passing rate distributions for study sites – counter at middle of passing facility**

### *5.4.3 Evaluation of Passing Related Parameters*

As an indicator of passing activity, normalised passing rate is preferable to passing rate but it is not ideal. This is because it is location specific and so it will tend to give high passing values for the shorter passing facilities. Furthermore, it doesn't measure the total passing activity for the whole facility but neither does percent passing at a location nor average percent passing. However, the parameter percentage of passing vehicles appears to be more sensitive to one-way flow than either passing rate or normalised passing rate.

### 5.5 Across Centreline Passing

The passing facilities in this study have an outer lane adjacent to an inner, passing lane for traffic travelling in one direction, the treated direction, and another lane for traffic travelling in the opposing direction. Some of the locations permit traffic travelling in the direction opposite to that of the passing lane to cross the centreline and use the passing lane against the primary direction of flow. This is to facilitate traffic travelling in the direction opposite to that of the passing lane to perform passing manoeuvres. A minor investigation of the extent of this type of "across centreline passing" was undertaken.

Two sites were inspected, site 2e and site 4j. For each site, for the traffic lane in the direction opposite to that of the passing lane, the series of traffic counters were arranged in the order they would be activated by traffic travelling through the site. Vehicle records at the first traffic counter were matched to vehicle records at the consecutive counter, and so on through the site, so that individual vehicles could be followed through the site. Vehicle speeds, vehicle types, and traffic counter activation timings identified where one vehicle passed another vehicle, and these passing instances were counted. The "across centreline passing" investigations were aided by some macros but were primarily conducted manually.

Some observations from the inspections are summarised in Table 12 below.

**Table 12: Passing by opposing direction vehicles**

| Site | Day | Approximate vehicles per day in direction opposite to that of the passing lane | Approximate number of "across centreline passes" | Approximate percentage of flow performing "across centreline passes" |
|---|---|---|---|---|
| 2e | 1 | 3120 | 36 | 1.2% |
| | 2 | 3240 | 37 | 1.1% |
| | 3 | 3210 | 41 | 1.3% |
| 4j | 1 | 4200 | 44 | 1.0% |
| | 2 | 4490 | 41 | 0.9% |
| | 3 | 4990 | 42 | 0.8% |

With reference to Table 12, the percentage of passing vehicles in the untreated direction, at about 1%, is negligible. It was also expected that the percentage of flow performing “across centreline passes” would be greater for passing facilities located on flat road gradient because of the longer sight distances available, but Table 12 shows little difference between flat and rolling road gradient.

Harwood et al. (1985) have derived a simple linear model from field data collected at 15 passing sites located over the United States for estimating “across centreline” passing rates (passes/hour/km) where passing in the opposing direction at a passing facility is permitted. This model applies to flow rates of between 50 and 400 vph and has a coefficient of determination ($r^2$), meaning 71% of the variation in observed "across centre line" passing rates can be explained by the model.

For a representative peak hourly flow of 350 vph, the model calculates an expected "across centreline" passing rate of 38.5 passes/hour/km. By comparison, the 0.8% to 1.2% "across centreline" passing observed at sites 2e and 4j correspond to rates of between 3 and 7 passes/hour/km for a peak hourly flow of 350 vph. Therefore, United States opposing direction passing rates appear to be 5 to 10 times greater than those of New Zealand.

This significant difference may be explained in part by both sites being located in flat upstream/downstream terrain thereby possibly providing good upstream and downstream opportunities to overtake rather than using the passing facility. Also the highly visible NZ Police presence at site 4j due to its targeting for mobile speed enforcement may have contributed to under-performance in both directions.

Another possible explanation is that New Zealand drivers are more reluctant to undertake passing in the opposing direction of a passing facility due to unfamiliarity with road rules or safety concerns, which may be justified or unjustified.

Further investigation of opposing direction passing rates at passing facilities where across centreline passing is permitted therefore merits further investigation given the large discrepancy between expected and observed opposing direction passing rates at sites 2e and 4j.

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# 6 Predictive Models for Operational Evaluation of Passing Facilities

Section 5 highlighted that the operational effectiveness of passing facilities varies, depending on traffic and geometric conditions. As a consequence, several predictive models have been developed, primarily by United States based researchers, using multiple regression analysis to allow these variations in effectiveness to be investigated as a function of geometric and traffic variables.

The availability of 72 hours of operational data at each of the six studied passing sites, covering traffic flows up to 808 vph, enabled these existing predictive models to be calibrated and modified for New Zealand conditions and also new models to be formulated as necessary.

## 6.1 Upstream – Downstream Reduction in Bunching

Two models have been proposed for predicting the difference in % following vehicles immediately upstream and downstream of a passing lane. The simpler model, presented in Harwood et al. (1985), when adjusted for metric units is:

$$\Delta PF = 7.64 - 0.04 FLOW + 0.45 UPF + 7.76 LEN \quad ...(6.1)$$

where:

- ΔPF = difference in percentage of following vehicles upstream and downstream of the passing lane based on 4-seconds headway criterion
- UPF = percentage of following vehicles upstream of the passing lane based on 4-seconds headway criterion
- LEN = length of the passing lane (km)
- FLOW = flow rate in treated direction (FLOW ≤ 400 vph)

This model was derived from 6 hours of operational data at 15 study sites and has a coefficient of determination ($r^2$) of 0.55. A positive value of ΔPF represents a reduction in % following vehicles.

A feature of equation 6.1 is that the negative sign of regression coefficient for flow rate implies an inverse relationship between flow rate and ΔPF. Although Harwood et al. (1985) considered this counterintuitive, it is consistent with the results tabulated in Table 9.

The second model, presented in Harwood and Hoban (1987), when adjusted for metric units is:

$$\Delta PF = -12.03 + 10.9\ln(LEN) + 0.0823 FLOW - \frac{471}{FLOW} + 9.59\ln(UPF) - 0.0247 \times FLOW \times \ln(UPF) \quad ...(6.2)$$

where variables are as previously defined for equation 6.1.

Equation 6.2 has been established from 85 computer simulation runs using the TWOPAS model. It has a high statistical confidence and illustrates the complexity of the relationships and interactions that influence the effectiveness of passing lanes. Equation 6.2 is valid for the range of passing lane lengths from 0.4 to 3.2 km, for a range of flow rates from 100 to 700 vph, and for a range of percentage of following vehicles upstream of the passing lane from 20 to 70%. Equation 6.2 can be used for passing lanes on highways with up to 30% heavy vehicles in the traffic stream in flat, moderately rolling or severely rolling terrain. However, equation 6.2 is not applicable to climbing lanes in mountainous terrain.

The variables in equations 6.1 and 6.2 were regressed against hourly data acquired for the passing lane sites (sites 2e, 3e, 4j, 5f and 6e) over the specified model ranges to establish if the model forms were appropriate for use under New Zealand conditions and if the model constants had to be modified. All the modelling results presented are statistically significant at the 95 percent level.

The derived model constant estimates and goodness of fit (coefficient of determination ($r^2$) and standard error of estimation (SE)) were as follows:

$$\Delta PF = -3.39 - 0.03 FLOW + 0.28 UPF + 4.28 LEN \quad ...(6.3)$$

($r^2$= 0.12, SE = 3.5, no. of observations = 179)

$$\Delta PF = -54.38 + 2.59\ln(LEN) + 0.125 FLOW + \frac{1580}{FLOW} + 13.37\ln(UPF) - 0.03 \times FLOW \times \ln(UPF) \quad ...(6.4)$$

($r^2$= 0.07, SE = 3.4 no. of observations = 192)

For equation 6.3, the most significant (p-value < 0.05) predictor variable was found to be UPF whereas for equation 6.4 it was ln(LEN).

Although the model forms were maintained, apart from a change in sign of the 1/FLOW term in equation 6.4, the fits were too low for use in investigating passing lane policy. Therefore, in an effort to find a model that explains more of the variation in ΔPF than equations 6.3 and 6.4, both average gradient (in %) of the passing facility (GPL) and percentage of light vehicle towing and heavy commercial vehicles in the traffic stream (LTHV) were added. The resulting models were:

$$\Delta PF = -9.76 - 0.04 FLOW + 0.39 UPF + 7.94 LEN + 0.57 GPL - 0.05 LTHV \quad ......(6.5)$$

($r^2$= 0.29, SE = 3.2, no. of observations = 179)

$$\Delta PF = -57.02 + 7.13\ln(LEN) + 0.007 FLOW + \frac{1405}{FLOW} + 15.92\ln(UPF) - 0.006 \times FLOW \times \ln(UPF) + 0.63 GPL - 0.11 LTHV \quad ...(6.6)$$

($r^2$= 0.31, SE = 2.9, no. of observations = 192)

Both models suggest that ΔPF has a positive relationship with gradient of the passing facility (i.e. increasing gradient assists operational effectiveness of a passing lane) and a negative relationship with LTHV (i.e. increasing % LVT & HCV reduces operational effectiveness of a passing lane), which seems intuitively correct.

Referring to Figures 17 and 18, which compares model predictions to observed ΔPF values, a trend can clearly be seen suggesting some predictive ability and so both models can be used for guidance. However, the degree of scatter about the regression line, at about ± 5 ΔPF, casts some questions about the use of either model to address questions of fundamental importance to

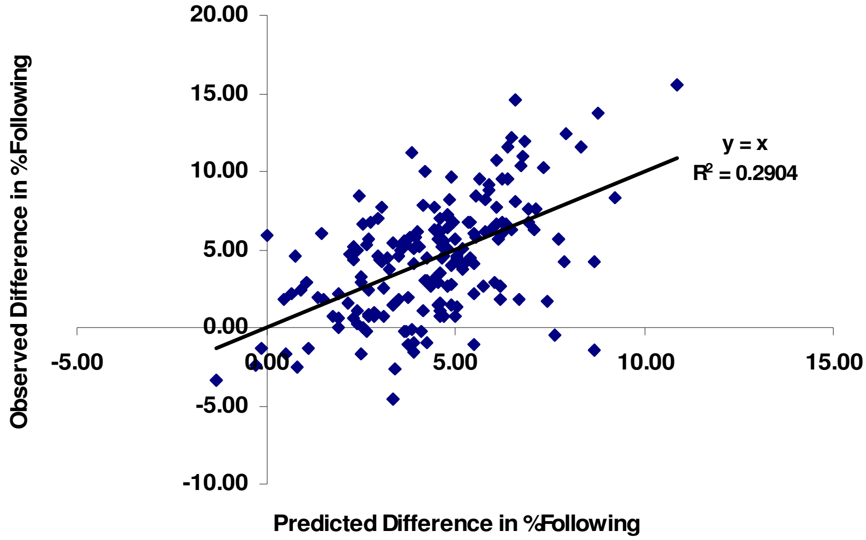


**Figure 17: Scatter plot of predicted (equation 6.5) versus observed difference in percentage of following vehicles immediately upstream and downstream of a passing lane for traffic flows 80 ≤ vph ≤ 400**

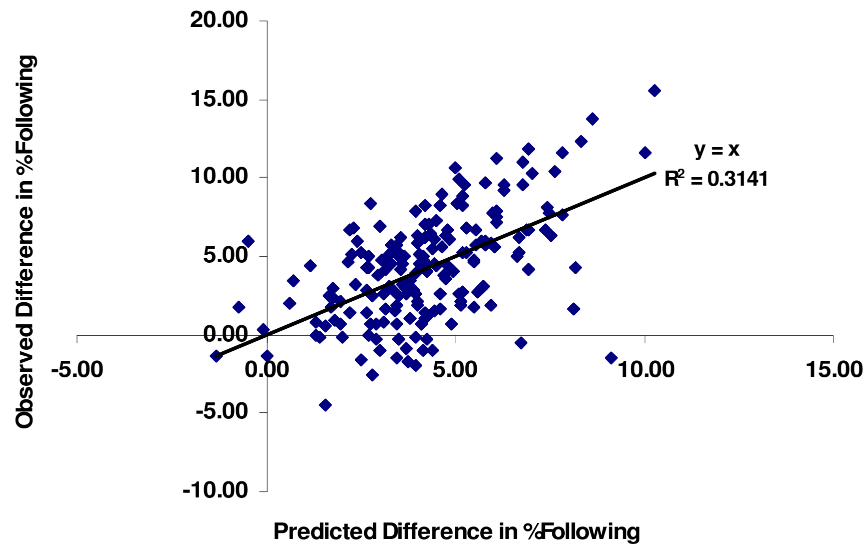


**Figure 18: Scatter plot of predicted (equation 6.6) versus observed difference in percentage of following vehicles immediately upstream and downstream of a passing lane for traffic flows 100 ≤ vph ≤ 700**

designers and policy makers such as the optimal passing lane length under different conditions of traffic and road gradient However, as trends may have been obscured by spurious events, the hourly data was "smoothed" by averaging over 50 vph bands. The effect of this data smoothing on the model that fitted the raw hourly data best (equation 6.6) was to improve the statistics of the model to an $r^2$ of 0.45 and standard error of 1.6 in predicted ΔPF. Also ΔPF shows significantly increased sensitivity to flow but decreased sensitivity to average gradient along the passing lane. The revised regression model using hourly data averaged over 50 vph intervals is:

$$\Delta PF = -69.98 + 5.70\ln(LEN) + 0.128FLOW + \frac{1688}{FLOW} + 18.2\ln(UPF) - 0.03 \times FLOW \times \ln(UPF)$$
$$+ 0.29GPL - 0.09LTHV \ldots (6.7)$$

($r^2$= 0.45, SE = 1.6, no. of observations = 32)

The most significant predictor variables (p-value < 0.05) in decreasing order of significance are ln(LEN), GPL, and ln(UPF).

The scatter plot of observed differences in % following vehicles upstream and downstream of the passing lane versus predictions from equation 6.7 is given in Figure 19. As expected, Figure 19 shows the observed and predicted values of ΔPF to be closer to each other than Figure 18.

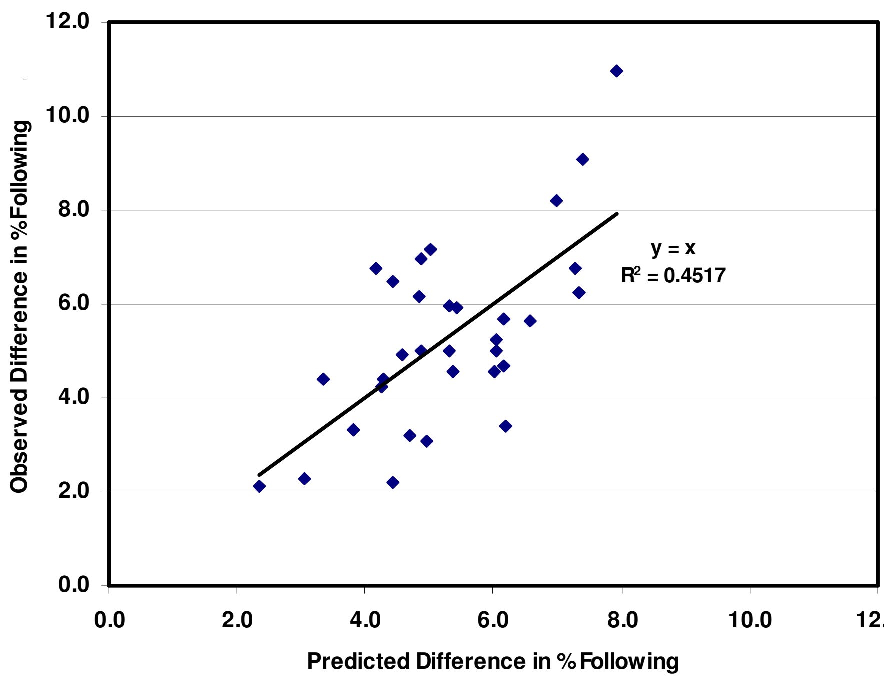


**Figure 19: Scatter plot of predicted (equation 6.7) versus observed difference in percentage of following vehicles upstream and downstream of a passing lane for traffic flows 100 ≤ vph ≤ 700**

## 6.2 Normalised Passing Rate

Harwood et al. (1985) found that passing rate had a strong relationship to flow rate. Their resulting regression model for predicting passing rate in the treated direction, when adjusted for metric units is:

$$NPR = 0.127FLOW - 6.02LEN + 1.35UPF \qquad for\ \ 50 \leq FLOW(vph) \leq 400 \ldots (6.8)$$

where NPR is normalised passing rate in passes per hour per kilometre and the other variables as previously defined in equation 6.1.

This model has a coefficient of determination ($r^2$) of 0.83. The model indicates that the normalised passing rate increases with increasing flow rate and with increasing % following vehicles upstream of the passing lane. The model also indicates that the normalised passing rate decreases with increasing passing-lane length.

Hourly passing rates obtained at the middle of the passing facility were aggregated over 50 vph flow intervals over a range from 50 vph to 400 vph for each of the five passing sites studied. The aggregated passing rates were then averaged and normalised prior to being regressed against the variables in equation 6.8. The resulting model constant estimates and goodness of fit (coefficient of determination ($r^2$) and standard error of estimation (SE)) were as follows:

$$NPR = -\,0.054FLOW - 24.44LEN + 1.67UPF \qquad for\ \ 50 \leq FLOW(vph) \leq 400 \ldots (6.9)$$

($r^2$= 0.36, SE = 19 no. of observations = 30)

The most significant predictor variables were found to be LEN (p-value 0.014) and UPF (p-value 0.016). It will be noted that the sign of the FLOW variable has changed, and only the UPF variable has a model constant that is of comparable value to that of equation 6.8.

Addition of the variable, average gradient (in %) of the passing facility (GPL), resulted in an improved regression model that explains 61% of the variance in the dependent variable, NPR. The revised model for normalised passing rate in the treated direction is:

$$NPR = 0.093FLOW - 27.74LEN + 0.51UPF + 3.47GPL \qquad for\ \ 50 \leq FLOW(vph) \leq 400 \ldots (6.10)$$

($r^2$= 0.61, SE = 15, no. of observations = 30)

Inclusion of the gradient term preserves the model form of equation 6.8 i.e. positive relationship between NPR and FLOW, which is not present in equation 6.9. Unfortunately, the influence of %LVT & HCV could not be investigated because of the aggregation process. The most significant predictor variables were GPL (p-value 0.0004) and LEN (p-value 0.001).

Generally better model fits are obtained with normalised passing rates (NPR) than with difference in % following vehicles upstream and downstream of the passing lane (ΔPL) ($r^2$= 0.61 c.f. $r^2$= 0.45). The results of Harwood et al (1985) also support the notion that predictive models based around normalised passing rate are more robust and so should be used in preference for assessing operational effectiveness of passing lanes.

A comparison of predicted and observed normalised passing rates over a 50 vph to 400 vph flow range is shown in Figure 20.

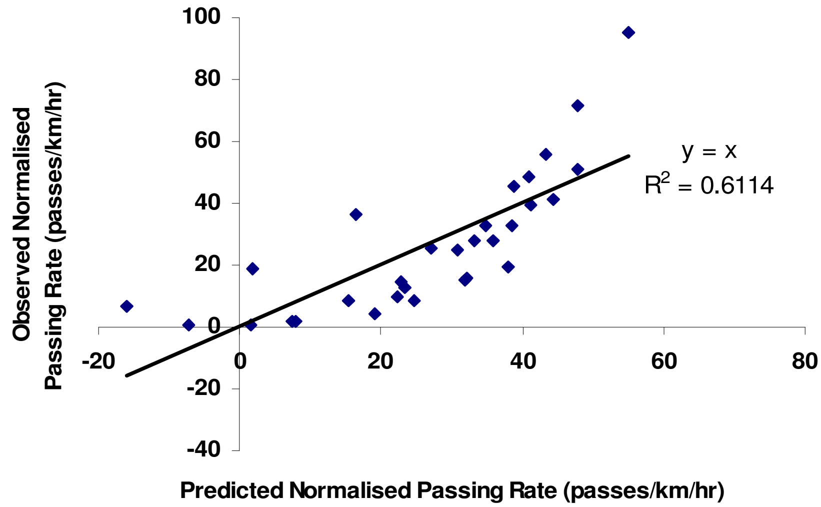


**Figure 20: Scatter plot of predicted (equation 6.10) versus observed normalised passing rate for traffic flows 50 ≤ vph ≤ 400**

### 6.3 Downstream Operational Length

As no models for predicting downstream operational length for passing lanes could be identified, hourly values of estimated operational length, derived using the procedure detailed in section 3 and based on % following vehicle distributions (2 seconds headway criterion), were regressed against geometric and traffic parameters over a 100 vph to 800 vph traffic flow range.

The model with the best fit was:

$$OL = 15.87 - 0.010FLOW - \frac{131.35}{DPF} + 0.047LTHV - 0.25GDS \ldots(6.11)$$

($r^2$= 0.17, SE = 3.8, no. of observations = 72)

where:

OL = downstream operational length (km)

FLOW = flow rate, vph, in treated direction (100 ≤ FLOW ≤800)

DPF = percentage of following vehicles immediately downstream of passing lane (11% ≤ DPF ≤ 40%)

LTHV = percentage of light vehicles towing and heavy commercial vehicles (5% ≤ LTHV ≤ 22%)

GDS = nominal downstream gradient in % (flat = 1.5%, rolling = 4.5%, and mountainous = 7.5%)

This model explains only 17% of the variation in the dependent variable (i.e. $r^2$ = 0.17) and the standard error of estimation is 3.8 km. The most significant predictors were 1/DPF (p-value 0.005) and FLOW (p-value 0.030).

The model shows downstream operational length to reduce with increasing flow, and downstream gradient and increase with increasing %LVT & HCV, which is as expected. However, increasing operational length with increasing % following vehicles immediately downstream seems counterintuitive and may be a consequence of the procedure adopted for deriving operational lengths.

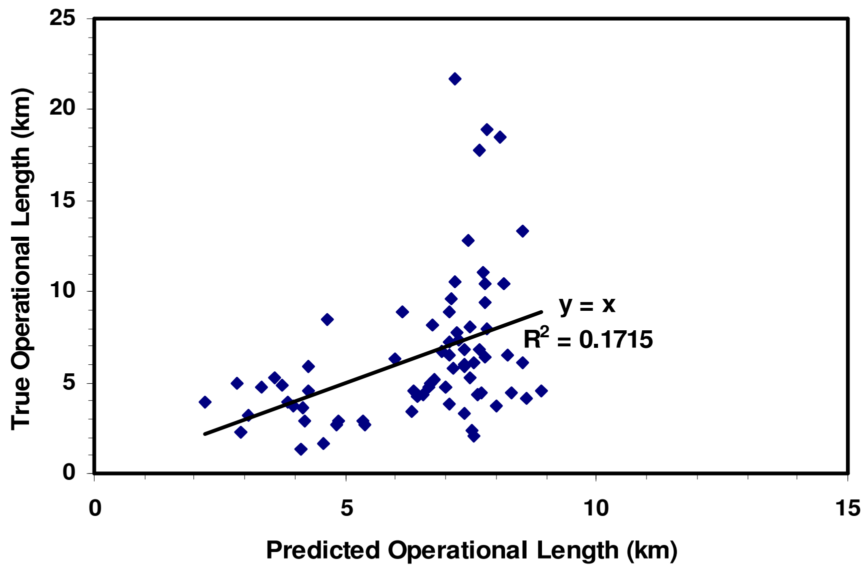


**Figure 21: Scatter plot of predicted (equation 6.11) versus derived downstream operational lengths for traffic flows 100 ≤ vph ≤ 800**

A scatter plot of model predictions versus corresponding derived downstream operational lengths is given in Figure 21. A strong linear trend is evident and it appears that four outliers, corresponding to very large derived downstream operational distances (> 17 km), are largely responsible for the low $r^2$ value.

From the data, there is no reason why such large operational distances should result. Therefore, to minimize the influence of spurious events and any random measurement errors, the hourly operational length data was averaged over 50vph bands and the regression analysis repeated. The model which resulted is given as equation 6.12 and its predictive capability illustrated in Figure 22.

$$OL = 21.29 - 0.017FLOW - \frac{221.25}{DPF} - 0.0025LTHV + 0.04GDS \ldots (6.12)$$

($r^2$ = 0.77, SE = 1.5, no. of observations = 14)

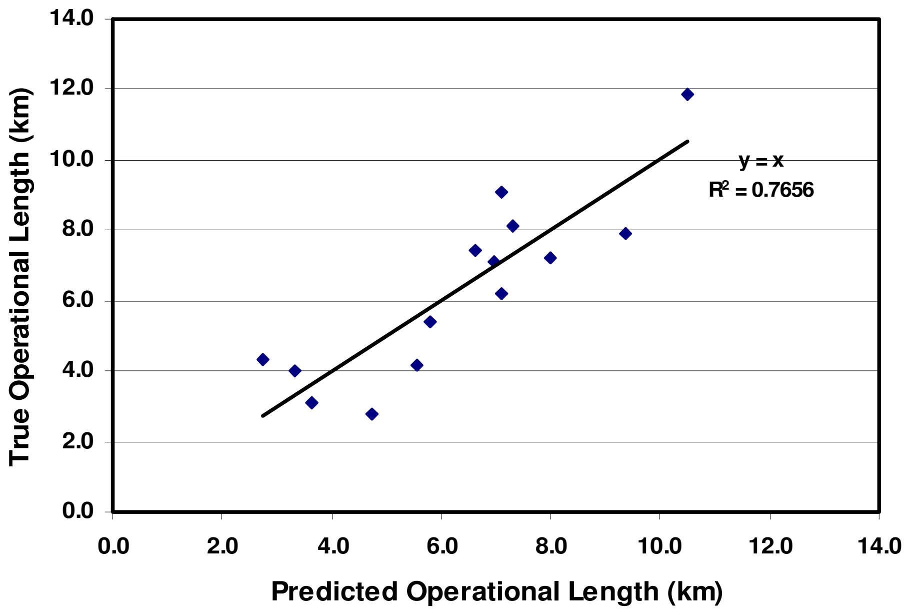


**Figure 22: Scatter plot of predicted (equation 6.12) versus derived downstream operational lengths for traffic flows 100 ≤ vph ≤ 800**

Comparing equation 6.12 to 6.11, it can be seen that the model form has changed, with the signs of variables LTHV and GDS reversing. Averaging increases the model's sensitivity to FLOW and 1/DPF while significantly reducing its sensitivity to percentage of light vehicles towing and heavy commercial vehicles (LTHV) and nominal downstream gradient (GDS). The most significant predictor variables become 1/DPF (p-value = 0.0010) and FLOW (p-value = 0.0025).

### 6.4 Exclusion of Site 4j Data

From discussions with NZTA Wanganui – Manawatu staff responsible for managing the section of SH 3 where the passing facility at site 4j is located, it was identified that there is a highly visible NZ Police presence in the vicinity of site 4j on account of this section of SH 3 being targeted for mobile speed enforcement. Therefore, regardless of NZ Police mobile speed enforcement operating at the time of the survey or not, the behaviour of the traffic would be affected outside of the operation periods as the route is regularly travelled as a rural commuter route and motorists would be aware of the possibility of NZ Police presence somewhere along the passing facility. The purpose of the speed monitoring programme is to increase the perception of being caught and so the driving speeds will be reduced outside of when there is a NZ Police presence. The modelling for the hourly data smoothed by averaging over 50 vph bands was therefore repeated with data for 4j excluded to establish if there were any significant changes to the model forms and fits.

The results were as follows:

***Upstream - Downstream Reduction in Bunching (c.f. equation 6.7)***

$$\Delta PF = -120.48 + 11.80\ln(LEN) + 0.013FLOW + \frac{3236.8}{FLOW} + 30.79\ln(UPF) - 0.009 \times FLOW \times \ln(UPF)$$
$$+ 0.82GPL + 0.018LTHV \ldots (6.13)$$

($r^2$= 0.74, SE = 1.3, no. of observations = 25)

The most significant predictor variables (p-value < 0.05) in decreasing order of significance remain ln(LEN), GPL, and ln(UPF).

***Normalised Passing Rate (c.f. equation 6.10)***

$$NPR = 0.063FLOW - 33.34LEN + 1.43UPF + 0.44GPL \quad for\ 50 \le FLOW(vph) \le 400 \ldots (6.14)$$

($r^2$= 0.93, SE = 12, no. of observations = 23)

The most significant predictor variables were LEN (p-value 0.000053) and GPL (p-value 0.03).

***Downstream Operational Length (c.f. equation 6.12)***

$$OL = 17.08 - 0.015FLOW - \frac{195.11}{DPF} + 0.177LTHV + 0.018GDS \ldots (6.15)$$

($r^2$ = 0.83, SE = 1.3, no. of observations = 13)

The most significant predictor variables become 1/DPF (p-value = 0.0014) and FLOW (p-value = 0.0031).

Exclusion of site 4j data produces a significant improvement in the fit of the model for estimating the difference in the percentage of following vehicles upstream and downstream of the passing lane ($r^2$ = 0.74) and even more so for the model for estimating normalised passing rate ($r^2$ = 0.93). However, there is only a very minor improvement to the fit of the model for estimating downstream operating length ($r^2$ = 0.83).

In all three cases the model form remains unchanged apart from the sign of the variable LTHV changing from negative to positive suggesting that the operational efficiency of a passing facility improves with increasing percentage of light vehicle towing and heavy commercial vehicles.

## 6.5 Remarks

The main objective of this study has been to investigate the operational effectiveness of selected passing sites that lie at the extremities of the NZ Transport Agency's long-term framework for passing and overtaking treatments. Accordingly, the operational data acquired is of limited scope for modelling purposes.

To enable robust modelling, a wider spread of sites throughout the framework would be required with at least 2-3 sites for each AADT/road gradient condition investigated so that atypical performance can be readily identified. Also a wider range of downstream conditions would need to be covered.

Despite the limitations of the existing modelling exercise, the following significant outcomes have emerged:

- Average gradient along a passing facility has a positive influence on the operational effectiveness of the passing facility i.e. increasing gradient assists passing manoeuvres within the facility.
- Of all the variables investigated, passing related measures, such as percentage of passing vehicles and passing rate, provided the most robust means for quantifying the operational effectiveness of passing facilities.
- Downstream operational length is particularly influenced by traffic flow and the percentage of following vehicles immediately downstream of the passing facility.
- The application of US based models for assessing the operational effectiveness of passing facilities to the six sites surveyed, showed the model forms to be appropriate but the estimates of passing performance were significantly greater than observed. This highlights the need to calibrate overseas derived models for local conditions.

# 7 Suitability of Study Sites

A comparative analysis of average percent passing values was undertaken to identify any peculiarity with the six passing sites selected for study. Percent passing was selected as on the basis of the modelling results summarised in Section 6 it is considered to be one of the more robust parameters for evaluating the performance of passing facilities.

Tables 13 -15 are two-way tables that show how the percent passing changes as a function of one-way flow and facility length for passing facilities located in mountainous, rolling and flat road gradient, respectively. These tables do not reflect how all passing facilities would perform but have been prepared to help understand how each of the study sites compares with similar lengths of passing facility in different terrain and how passing facilities with the same road gradient compare over different passing lengths. The estimated percent passing value is an average value based on counter locations within the passing length rather than at a single specific location.

**Table 13: Estimated average percent passing for average gradient along passing facility > 6%**

| One-Way Flows (vph) | Estimated Percentage Passing (%) for Specific PL Lengths (m) | | | | | |
|---|---|---|---|---|---|---|
| | 325 m (8j) | 600 m (2e) | 800 m | 1000 m | 1200 m (6e) | 1400 m |
| 100 | **16** | **12** | 11* | 9* | **7** | 5* |
| 200 | **25** | **17** | 17* | 14* | **12** | 9* |
| 400 | - | - | - | - | **17** | 17* |
| 700 | - | - | - | - | **23** | 29* |
| **NOTES** (1) Extrapolated/interpolated values are shown as *. (2) Study site ID given in brackets. | | | | | | |

**Table 14: Estimated average percent passing for average gradient along passing facility 3% - 6%**

| One-Way Flows (vph) | Estimated Percentage Passing (%) for Specific PL Lengths (m) | | | | | |
|---|---|---|---|---|---|---|
| | 325 m | 600 m (3e) | 800 m | 1000 m | 1200 m | 1400 m |
| 100 | - | **12** | 11* | 9* | 8* | 7* |
| 200 | - | **13** | 13* | 12* | 12* | 12* |
| 400 | - | 15* | 16* | 17* | 17* | 18* |
| 700 | - | - | - | 24* | 26* | 30* |
| **NOTES** (1) Extrapolated/interpolated values are shown as *. (2) Study site ID given in brackets. | | | | | | |

**Table 15: Estimated average percent passing for average gradient along passing facility < 3%**

| One-Way Flows (vph) | Estimated Percentage Passing (%) for Specific PL Lengths (m) | | | | | |
|---|---|---|---|---|---|---|
| | 325 m | 600 m | 800 m | 1000 m (4j) | 1200 m | 1400 m (5f) |
| 100 | **-** | - | - | **2** | 6* | **9** |
| 200 | **-** | - | - | **3** | 9* | **15** |
| 400 | - | - | - | **7** | 14* | **21** |
| 700 | - | - | - | 12* | 22* | 32* |
| **NOTES** (1) Extrapolated/interpolated values are shown as *. (2) Study site ID given in brackets. | | | | | | |

For sites 2, 3 and 8, the average percent passing values are the average of values surveyed 30 m downstream of the start and 30 m upstream of the end for the passing facility. For site 4, only the mid-point and end values were used because of equipment malfunction. For sites 5 and 6, the average of three or more locations was taken, involving the start, end and set intervals along the passing length. The surveyed percentage passing values are tabulated as bolded shaded values.

The only direct comparison possible between Tables 13, 14 and 15 is for the 600 m long passing lanes (sites 2e & 3e). Therefore, interpolated and extrapolated values have been used to help understand and identify underlying patterns between the study sites. Comparing sites 2e and 3e, a similar result was obtained for both facilities at about 100 vph. At higher flows, site 2e had a higher and more consistent upstream demand compared to site 3e.

Table 14 is the most sparsely populated with surveyed values. As the 1400 m values in Tables 13 and 15 are consistent, they were linearly interpolated to obtain 1400 m values for Table 14. This in turn allowed the 800 m – 1200 m values to be estimated. However, extra survey data is needed to confirm the estimated trends within Table 14.

A marked increase in percent passing is shown in Tables 15 over a relatively narrow range of passing lane lengths (1-1.4 km). Given that site 6e (1200 m) has similar values to site 5f (1400 m), the most likely explanation is that site 4j is under-performing. However, the interpolated 1200 m values in Table 15 are similar to the surveyed 1200 m values (site 6e) in Table 13, especially at higher flows (400-700 vph one-way). Therefore, passing facilities on flat gradient may be more sensitive to passing length. Further research is suggested.

Taken overall, Tables 13, 14 and 15 show that for a specified passing facility length, the efficiency of the passing facility increases with increasing one-way flow irrespective of road gradient. For one-way flows below 200 vph, there is an indication that passing facilities in mountainous and rolling road gradient have higher passing activity relative to length as they become shorter. Conversely, for one-way flows greater than 400 vph, passing facilities show higher passing activity as they become longer irrespective of road gradient. Passing activity on shorter passing facilities tends to plateau at higher flows.

# 8 Policy and Operational Field Data Comparisons

## 8.1 Choice of Parameters

The only direct comparisons that can be made between the NZ Transport Agency's proposed strategy for passing and overtaking treatments and operational data are on the basis of passing facility length and downstream operational length, which equates to spacing from the end of one passing lane to the start of the next in the same direction. A tabulated summary of this comparison is provided in Table 16.

For the long-term Policy framework, projected AADT has been used and is based on a directional split of 55%/45% during peak hours and the critical peak hour volume is assumed to be 10.5% of AADT. About 95% of all hourly flows within the year would be less than 10.5% AADT (Land Transport NZ, 2007). Both estimated projected flows and projected AADT intervals tabulated in Table 16 have been derived on this basis.

With reference to Table 16, it will be noted that the observed range of downstream operational lengths span the spacings specified in the policy framework. It also appears that the passing lane lengths in the policy framework are generous, particularly for rolling road gradient (all traffic volumes) and mountainous road gradient at low traffic volumes. This may be because the operational data may reflect very favourable circumstances for these cases.

It is clear from Table 16 that the operational performance of passing facilities is very much influenced by a number of factors, as shown by the wide range of downstream operational lengths, particularly for passing lanes with flat downstream gradients and good passing opportunities.

The research findings suggest that downstream operational length is influenced by flow rate in direction of travel, downstream road gradient, downstream passing opportunities, percentage of light vehicles towing and heavy commercial vehicles in the traffic stream and possibly % following vehicles immediately downstream of the passing facility.

By comparison, optimum passing lane length is influenced by flow rate in direction of travel, percentage of light vehicles towing and heavy commercial vehicles (%LVT & HCV) in the traffic stream and headway distributions immediately upstream of the passing facility. Generally, models derived from the field data indicate that passing lane length should decrease with increasing average gradient along the passing lane and increase with increasing %LVT & HCV in the traffic stream.

While the policy framework takes into account flow rate and gradient through the terrain classifications of flat, rolling and mountainous, no guidance is given in regard to % LVT & HCV in the stream. The results from the modelling suggests that when going from low %LVT & HCV to high %LVT & HCV, passing lane length should increase and spacing decrease by about 15% to 20% in both cases. More research is required to better quantify these effects, which from the modelling are expected to happen at the same time.

It is unrealistic to expect the policy framework to account for the other important factors such as headway distribution and passing sight distances. However, if there is a weakness with the proposed policy, it is that the projected AADT based traffic categorisation is too coarse. Therefore,

## Table 16: Operational Data Comparisons with Proposed Policy (based on 2-seconds headway criterion)

| Site Details | | | Survey Results | | | | | | Proposed Policy | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Site ID** | **PL or SVB Average Gradient (%)** | **Down-Stream Road Gradient (%)** | **One-Way Flow (vph)** | **Estimated Projected AADT [1] (vpd)** | **LVT & HCV (%)** | **PL or SVB Length (excl. tapers) (km)** | **Downstream Operational Length (km) Range** | **Downstream Operational Length (km) Average** | **Projected AADT Interval (vpd)** | **PL or SVB Length (excl. tapers) (km)** | **Spacing (km)** | **Comments** |
| 2e | 6.8 | Flat (< 3%) | 261-328 | 4,500-5,700 | 8-13 | 0.599 | 4.1-11 | 6.8 | 4,000-5,000 | 1.2 | 5 or 10 | 10 km spacing OK |
| | | | 119-250 | 2,100-4,300 | 9-14 | | 2.1-10.5 | 5.3 | 2,000-4,000 | 0.6-0.8 | 10 | Higher flow more critical. 10 km spacing OK if 0.8 km PL used. |
| 3e | 5.7 | Mountainous (> 6%) | 190-250 | 3,300-4,300 | 17-21 | 0.556 | 6.4-13.7 | 10.0 | 2,000-4,000 | 0.6-0.8 | 10 | 0.6-0.8 km PL @ 10 km spacing OK. |
| | | | 117-188 | 2,000-3,300 | 13-19 | | 2.3-18.9 [2] | 6.3 [2] | 2,000-4,000 | 0.6-0.8 | 10 | Higher flow more critical. 10 km spacing OK. |
| 4j | 0.4 | Flat (< 3%) | 343-487 | 5,900-8,400 | 5-10 | 0.939 | 3.4-21.7 [3] | 13.5 [3] | 5,000-7,000 | 1.2 | 5 or 10 | Within transition range. Use lower treatment layout. 10 km spacing OK. |
| 5f | 0.27 | Rolling (3% - 6%) | 355-558 | 6,100-9,700 | 13-20 | 1.397 | 3.4-12.8 | 6.9 | 7,000-10,000 | 1.5 | 5 or 10 | 5 or 10 km spacing OK. |
| 6e | 7.2 | Mountainous (> 6%) | 693-805 | 12,000-13,900 | 7-13 | 1.192 | 1.3-5.4 | 3.9 | 10,000-25,,000 | 1.2-1.5 | 5 | 5 km spacing OK. Consider crawler lanes for higher flows. |
| | | | 530-680 | 9,200-11,800 | 11-13 | | 1.6-5.9 | 3.2 | 7,000-10,000 | 1.2 | 5 | Within transition range. Use lower treatment layout. Higher flow more critical so 5 km spacing OK. Consider crawler lanes. |
| 8j | 6.4 | Mountainous (> 6%) | 170-192 | 2,900-3,300 | 14-15 | 0.325 | 1.2-3.6 | 2.4 | 2,000-4,000 | 0.6-0.8 | 10 | 0.6-0.8 km PL @ 10 km spacing OK if OT treatments also used. |
| | | | 114-143 | 2,000-2,500 | 12-18 | | 3.5-14.5 | 8.4 | 0-2,000 | 0.6-0.8 PL<br>0.325 SVB | 10<br>10 | 0.325 km SVB` @ 10 km close enough. |

NOTES:

1. Based on 55/%45% directional split and peak hourly flow of 10.5% of AADT (approx. $125^{th}$ percentile peak hour flow).
2. From Figure 21, site 3e data contains outlier point of 18.9 km.
3. From Figure 21, site 4j data contains outlier points of 17.8, 18.5 and 21.7 km. When mathematical model for operational length with 50 vph bands was used, an operational length of 7.6 km was calculated. Site 4j is on urban fringe with 65%/35% directional split and peak one-way flow is 11% of AADT. Therefore surveyed downstream operational length should be longer than for rural strategic non-recreational route.

consideration should be given to supplementing the AADT categorisation within the long-term framework with directional hourly flows and if possible % LVT & HCV in the traffic stream.

However, any increase in %LVT & HCV would probably be less marked for flat and rolling road gradients but could affect passing lanes in mountainous road gradient or if passing lane lengths are consistently shorter along a road section compared to the Passing and Overtaking Policy's long-term framework. Therefore, rather than altering the projected AADT categories for mountainous gradients, possibly future provision for crawler shoulders or lanes should be considered.

## 8.2 Conditions Affecting Individual Site Results

For site 4j, an average downstream operational length of 13.5 km was obtained but this is based on data from counters located at 0.9 and 3.3 km downstream of the passing lane, as other counters may be affected by the downstream effect of SH 54 joining SH 3. The value of 7.6 km based on mathematical modelling would seem a better estimate of downstream operational length. As this site is located on an urban fringe route, there is less opposing flow and therefore the 7.6 km calculated operational length is longer than would be expected for a rural strategic non-recreational route with the same peak hourly flow in the treated direction.

Harwood and Hoban (1987) outlines Canadian research that identified a relationship for estimating what proportion of an hour gaps in the opposing traffic are greater than 25 seconds. Gaps greater than 25 seconds are considered suitable for overtaking. This relationship is given as equation 8.1 and can be used as an initial guess at the effect of differing opposing flows, though it should be verified under New Zealand conditions.

$$proportion\ of\ hour\ with\ gaps > 25\sec s = e^{-0.0018626 \times OFLOW} \quad ...(8.1)$$

where: OFLOW = opposing traffic flow (vph)

The proportion of available sight distance is fixed under both opposing flows. It is also assumed that there will be a large amount of overtaking sight distance along the road section, as the terrain around site 4j is mainly flat. For 400 vph peak one-way flow, the opposing flows are expected to be about 215 vph (rural urban fringe) and 330 vph (rural strategic non-recreational). There could be 0 – 30% (i.e. 0.35/0.45 from section 5.1.4) less overtaking opportunities if site 4j was on a rural strategic non-recreational route. This reduction could equate to a downstream operational length of about 5.3 km - 7.6 km (i.e. (0.7- 1) x 7.6 km), which compares favourably with the Policy framework of 1.2 km passing lanes at 10 km spacings (with 5 km spacings in some high demand areas).

The regular targeting of site 4j by NZ Police for mobile speed enforcement would have a marked effect on its performance. This in turn makes it difficult to determine whether shorter passing lanes on flat terrain, such as site 4j (0.93 km), are prone to underperforming or whether operating speeds are sufficiently high and there are enough available overtaking opportunities along the road section that vehicle drivers are not as inclined to use the passing lane compared to other sites with less favourable road conditions. Regardless of these two scenarios, as the opposing traffic volume increases, drivers would become more reliant on passing facilities to provide passing opportunities.

At site 3e, for periods of hourly one-way flow where the upstream demand was consistently increasing, the percent passing values were similar to site 2e at the same one-way flow.

Site 6e is also on a rural urban fringe route but the directional split is similar to sites 2e, 3e, 5f and 8j and therefore the downstream operational length inferred from the survey results should be similar to a rural strategic non-recreational route.

Site 5f has a passing lane with about 0.27 % road gradient and the upstream and downstream road gradient is categorised as rolling. Site 5f with its flat gradient would suggest that not all passing lanes on flat gradients under-perform to the same extent as site 4j.

Site 5f had an average downstream operational length of about 3.9 km. If a 1.5 km passing lane was provided on a 3-6% road gradient, as indicated within the Policy framework, the downstream operational length is expected to be longer, which would be in line with the 5 km spacing under the Policy framework.

In summary, the research indicates that the long-term framework has passing facility spacings that are appropriate. There also appears to be a degree of “future proofing” in the passing facility lengths, which is desirable to take into account other influences, such as possible changes in the proportion of LVT and HCV traffic.

# 9 Conclusions and Recommendations

Based on the data collected during three day continuous classifier surveying at 5 passing lanes and one slow vehicle bay, the following conclusions and associated recommendations have been derived.

***Long-Term Policy Framework***

1. A comparison of the NZ Transport Agency's proposed strategy for passing and overtaking treatments with the acquired operational data confirmed that the long-term framework has passing facility spacings reasonably correct and that there appears to be a degree of "future proofing" in the passing facility length to take into account other influences, such as possible increases in heavy commercial traffic, which will require passing facilities to be longer.

2. The structure of the policy, which is based around projected AADT (used to reflect one-way hourly flow) and road gradient (used to reflect upstream demand, average gradient on the passing facility and possibly demand immediately downstream of the facility), was also shown to be correct as these two parameters significantly influence operational effectiveness.

3. Crawler shoulders and crawler lanes appear to have been omitted as treatments in the long-term framework. However, the survey results indicate that these treatments could be included for mountainous road gradients to provide consistency with other parts of the NZ Transport Agency's Passing and Overtaking Policy.

4. Preliminary modelling of the study sites indicated that: i) passing length, ii) upstream demand, iii) average gradient of passing facility, iv) hourly one-way flow and v) demand immediately downstream of the passing facility are significant predictor variables affecting the passing facility's effectiveness. Another important variable, but not identified as a significant predictor variable, was the percentage of light towing and heavy vehicles.

5. The interaction between the above-mentioned significant predictor variables is complex and further research would be required before inclusion within the long-term framework. However, ranges for key influences, such as one-way flow, percentage of light and heavy vehicles and if possible upstream demand should be considered when applying the long-term framework. If unusual conditions were identified, the Policy layout could be altered to suit, taking into account the above-mentioned significant predictor variables.

***Performance Parameters***

6. Of all the variables investigated, passing related measures, such as percentage of passing vehicles and normalised passing rate, appeared to provide the most robust measure of operational effectiveness of passing lanes and so their use is recommended in any further studies of passing facilities.

7. As an indicator of passing activity, normalised passing rate is preferable to passing rate but it is not ideal. This is because it is location specific and so it will tend to give high passing values for the shorter passing facilities. Furthermore, it doesn't measure the total passing activity for the whole facility but neither does percent passing at a location nor average percent passing.

However, the parameter percentage of passing vehicles appears to be more sensitive to one-way flow than either passing rate or normalised passing rate

8. For a specified passing facility length, the percentage passing increases with increasing one-way flow irrespective of road gradient. For one-way flows below 200 vph, there is an indication that there is more passing activity on passing facilities in mountainous and rolling road gradient. For one-way flows greater than 400 vph, passing facilities have more passing activity as they become longer, irrespective of road gradient.

9. Per kilometre of facility, the most effective with respect to passing rates are slow vehicle bays, followed by short passing lanes, with long passing lanes being the least effective. This suggests that more short passing lanes would be more effective than fewer long passing lanes. However, treatments with shorter passing lengths have less traffic flow capacity and so their service life is limited. Therefore, shorter treatments are only suitable over lower traffic ranges.

10. The passing rate was shown to increase with increasing traffic flow. Up to 200 vehicles per hour (one-way), the passing rate is fairly constant throughout the length of the passing facility. Above this flow rate, the highest passing rates occur near the middle of the facility for short passing lanes and a quarter of the way down the facility for long passing lanes.

11. At higher directional flows, the downstream operational length of passing lanes decreases with increasing traffic volume and increasing percent following.

12. The downstream effectiveness of a passing facility declines as the headway increases. Typically, for the same hourly traffic flow, the downstream operational length derived for a 2-seconds headway is between 1.1 and 2 times that calculated for a 4-seconds headway.

13. When applied to the surveyed passing facilities, overseas models overestimated their operational effectiveness in terms of passing rates and reduction in percent following. This highlights the need to calibrate overseas derived models for local conditions.

14. Regression modelling was applied to operational data acquired over a 72 hour period at each of the six sites surveyed. Traffic flows up to 808 vph were covered. The regression modelling showed operational effectiveness of a passing facility to be strongly related to traffic flow, road gradient in the vicinity of the passing facility, and percentage of light vehicles towing and heavy commercial vehicles in the traffic stream.

***Further Investigation***

15. Immediately downstream of the passing facility, the reduction in percentage of following vehicles based on a 4-seconds headway criterion was 4.4 percent. However, there was an indication that the difference in percentage of following vehicles upstream and downstream of the passing facility reduces with increasing flow for both 2 and 4-seconds headway. This merits further investigation as many factors could cause this situation such as percentage of following as a function of traffic flow and downstream conditions near to the passing lane taper.

16. “Across centreline” passing rates observed where passing in the opposite direction at a passing facility is permitted was minimal at 0.8% to 1.2% corresponding to 3 and 7 passes/hour/km for a peak hourly flow of 350 vph. The layout of counters within the study was

not conducive to a detailed study of overtaking behaviour in the opposite direction. However, as this rate of overtaking in the opposite direction is markedly lower than expected from overseas research, further investigation is merited to establish the cause.

17. Given the quality of the database that has been generated, it is recommended that additional analysis involving vehicle speeds and downstream horizontal and vertical sight distances should be undertaken to see if regression models can be formulated that better explain the variances observed in the operational effectiveness of the six passing facilities surveyed.

18. Additional sites could be investigated to verify the Policy framework over a greater range of traffic flows and road gradients and to improve the robustness of mathematical models derived for predicting the operational effectiveness of passing facilities.

Report Authored By:


| | |
|---|---|
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| Research Engineer | Research Manager |
| Transport & Environmental Sciences | Physical & Engineering Sciences |

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## 11 Acknowledgements

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# Appendix A: Study Sites

## A1 Information Provided

The following information is provided for each of the passing sites studied:

1. Summary of geometric and traffic characteristics.
2. Schematic of the placement of the automated (Metro-Count 5600) vehicle classifiers with their linear location and measured daily traffic flows superimposed.
3. Spatial location of the automated (Metro-Count 5600) vehicle classifiers.
4. GPS co-ordinates of the automated (Metro-Count 5600) vehicle classifiers.
5. Panoramic photographs of the start, middle and end of the passing zone.
6. Plot of the proportion of vehicles with headway ≤ 4 seconds at vehicle classifiers upstream and downstream of the passing facility.
7. Plot of the variation in proportion of vehicles with headway ≤ 4 seconds upstream and downstream of the passing facility with linear distance.

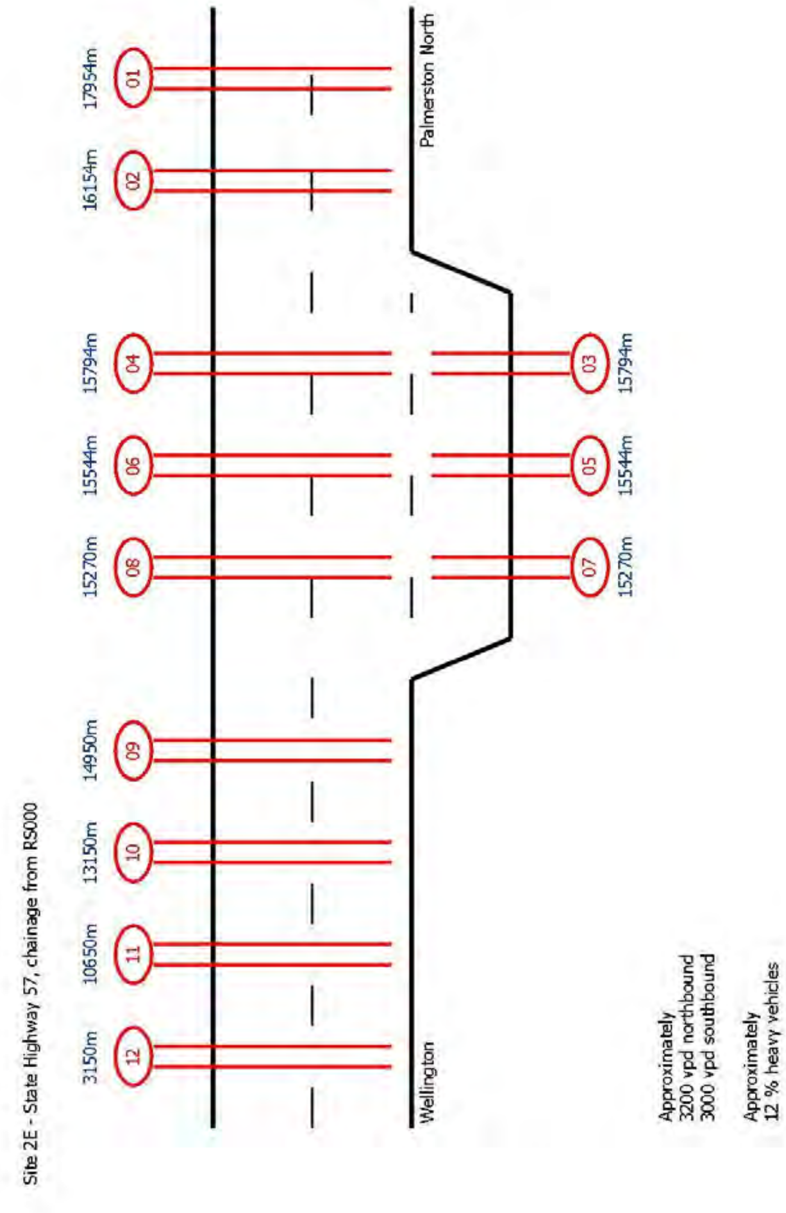
***Note:*** *For site 4j, the above information is provided for passing facilities in the deceasing and increasing directions, although only data from the increasing direction was used in investigating correlations between observed and predicted operational effectiveness.*

## A2 Short Passing Lane, Site 2e

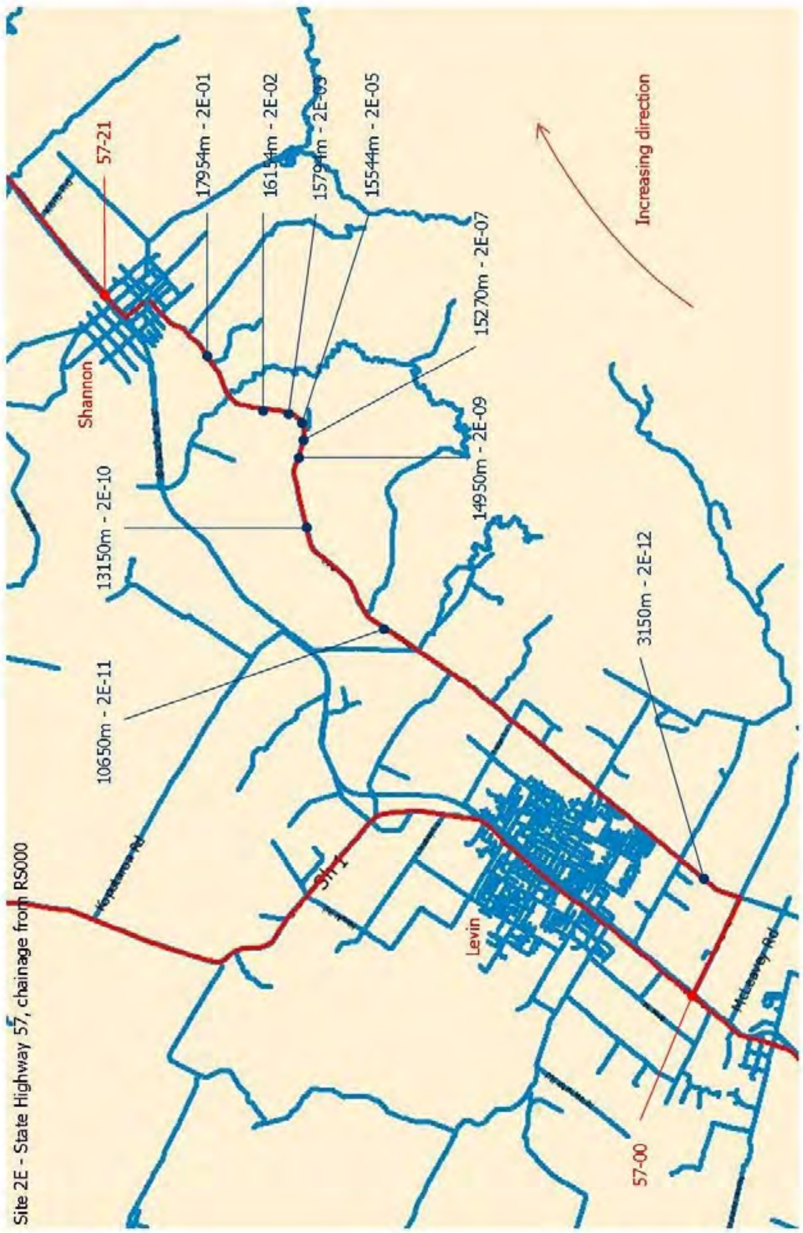
### *A2.1 Characteristics*

Nominal Length of Passing Lane: 646 m
Terrain: Flat
Average Gradient of Passing Lane: 6.8%
RAMM AADT: 8127
Measured (3 day average) AADT: 6200
RAMM % HCV: 11%
Measured (3 day average) % HCV: 12%
Monitoring Period: 17/07/2007 to 19/07/2007

### A2.2 Layout of Automated Classifiers



### A2.3 Spatial Location of Automated Classifiers



### *A2.4 GPS Co-Ordinates of Automated Classifiers*

| Counter Number | Displacement (m) | Latitude | Longitude | Direction A>B | Direction B>A | Comments |
|---|---|---|---|---|---|---|
| 1 | SH57-RS0/ 17954 | S40,33.6207 | E175,24.2784 | North Bound | South Bound | |
| 2 | 16154 | S40,34.2617 | E175,23.4572 | North Bound | South Bound | |
| 3 | 15794 | S40,34.4517 | E175,23.4256 | South Bound | N/A | Single Lane |
| 4 | 15794 | S40,34.4521 | E175,23.4167 | North Bound | South Bound | |
| 5 | 15544 | S40,34.5855 | E175,23.3939 | South Bound | N/A | Single Lane |
| 6 | 15544 | S40,34.5819 | E175,23.3820 | North Bound | South Bound | |
| 7 | 15270 | S40,34.6985 | E175,23.2572 | South Bound | N/A | Single Lane |
| 8 | 15270 | S40,34.6907 | E175,23.2534 | North Bound | South Bound | |
| 9 | 14950 | S40,34.7078 | E175,23.0329 | North Bound | South Bound | |
| 10 | 13150 | S40,34.7688 | E175,21.7990 | North Bound | South Bound | |
| 11 | 10650 | S40,35.6093 | E175,20.4953 | North Bound | South Bound | |
| 12 | 3150 | S40,38.8522 | E175,17.3299 | North Bound | South Bound | |

### A2.5 Panoramic Views

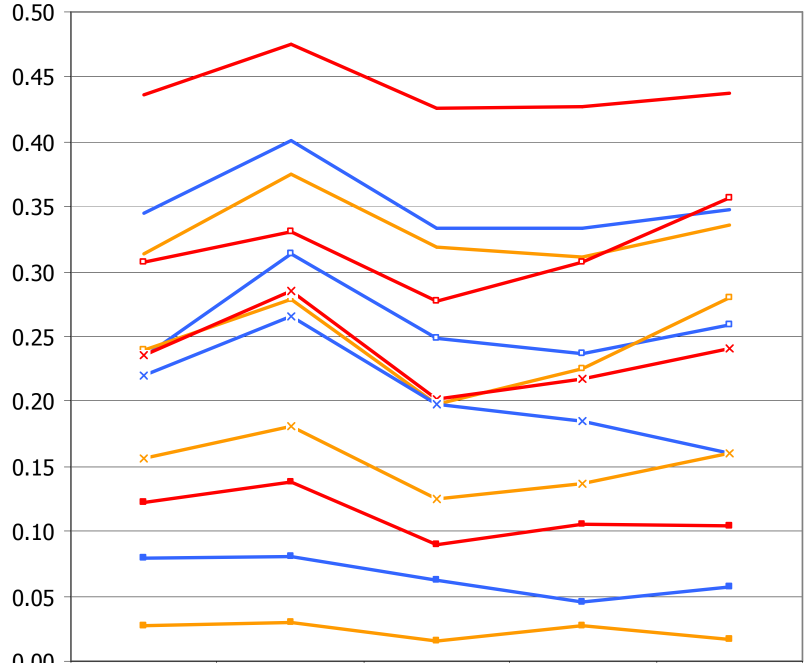
**Start**

**Middle**

**End**

### *A2.6 Proportion of vehicles with headway ≤ 4 seconds upstream and downstream of the passing facility at counter locations, treated direction.*

**Site 2E:** Proportion with headway <=4.0s (bunched), by vehicles per hour
(Passing lane between Counter 2 and Counter 9)

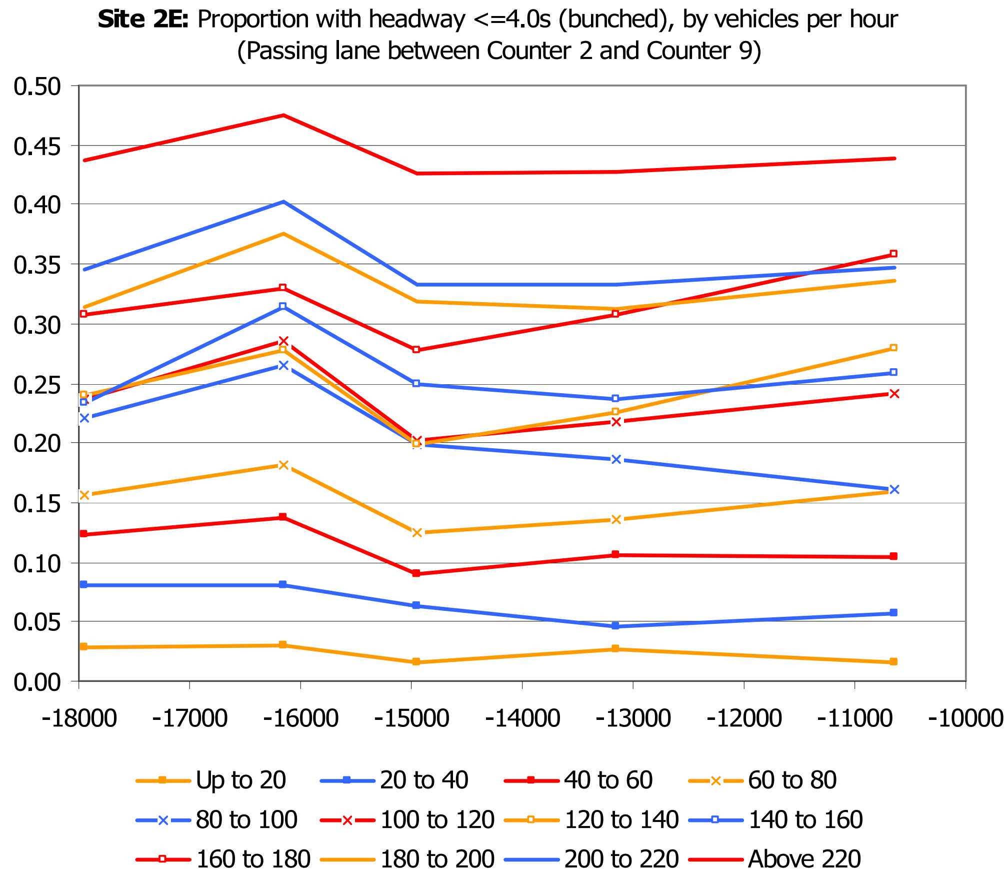



| | Counter 1 | Counter 2 | Counter 9 | Counter 10 | Counter 11 |
|---|---|---|---|---|---|
| Above 220 | 0.4364 | 0.4748 | 0.4252 | 0.4272 | 0.4379 |
| 200 to 220 | 0.3448 | 0.4015 | 0.3332 | 0.3334 | 0.3473 |
| 180 to 200 | 0.3132 | 0.3751 | 0.3184 | 0.3115 | 0.3356 |
| 160 to 180 | 0.3079 | 0.3301 | 0.2770 | 0.3068 | 0.3574 |
| 140 to 160 | 0.2340 | 0.3133 | 0.2491 | 0.2366 | 0.2586 |
| 120 to 140 | 0.2402 | 0.2780 | 0.1982 | 0.2252 | 0.2795 |
| 100 to 120 | 0.2360 | 0.2847 | 0.2018 | 0.2174 | 0.2412 |
| 80 to 100 | 0.2202 | 0.2653 | 0.1984 | 0.1854 | 0.1602 |
| 60 to 80 | 0.1567 | 0.1810 | 0.1247 | 0.1361 | 0.1597 |
| 40 to 60 | 0.1225 | 0.1377 | 0.0901 | 0.1053 | 0.1036 |
| 20 to 40 | 0.0800 | 0.0807 | 0.0628 | 0.0450 | 0.0574 |
| Up to 20 | 0.0279 | 0.0300 | 0.0153 | 0.0275 | 0.0164 |

Counter Distance Key:

| Counter No: | RS (km) | Distance from Counter 1 (km) |
|---|---|---|
| 1 | 17.954 | - |
| 2 | 16.154 | 1.800 |
| 9 | 14.950 | 3.004 |
| 10 | 13.150 | 4.804 |
| 11 | 10.650 | 7.304 |

### A2.7 Variation in proportion of vehicles with headway ≤ 4 seconds with distance, treated direction.

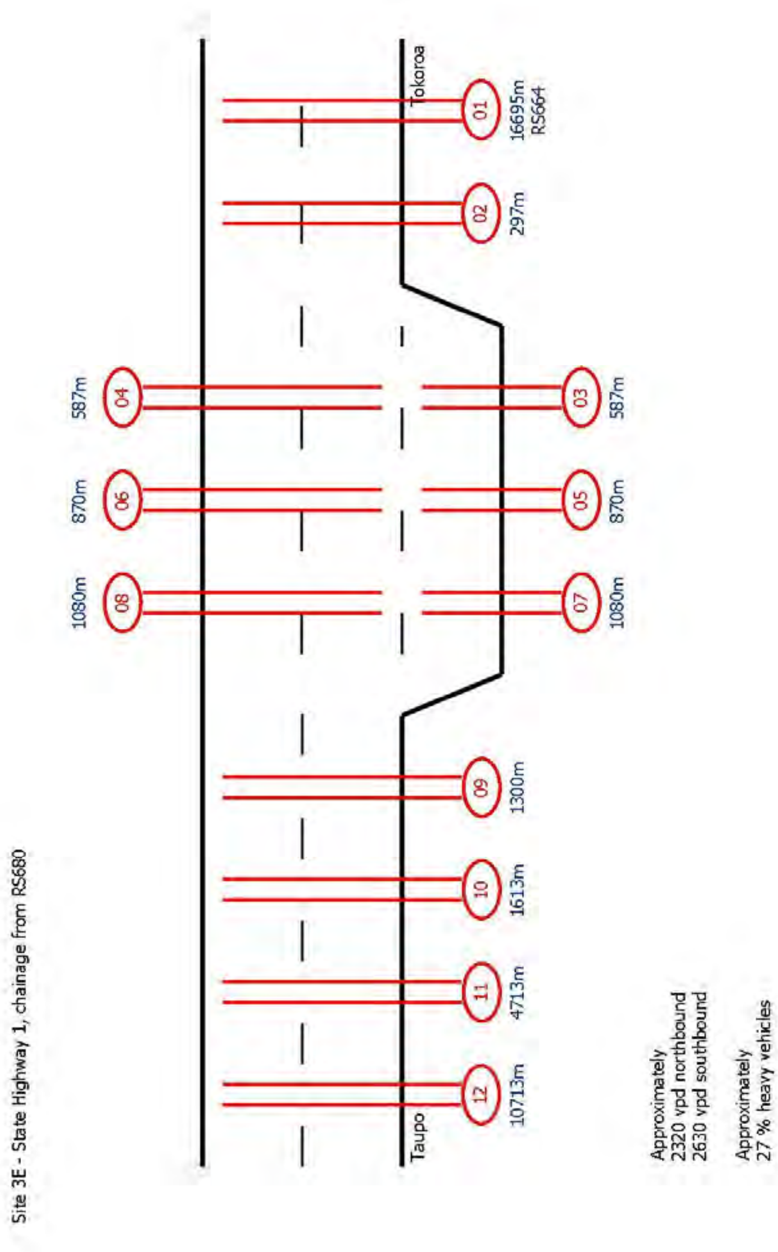


## A3 Short Passing Lane, Site 3e

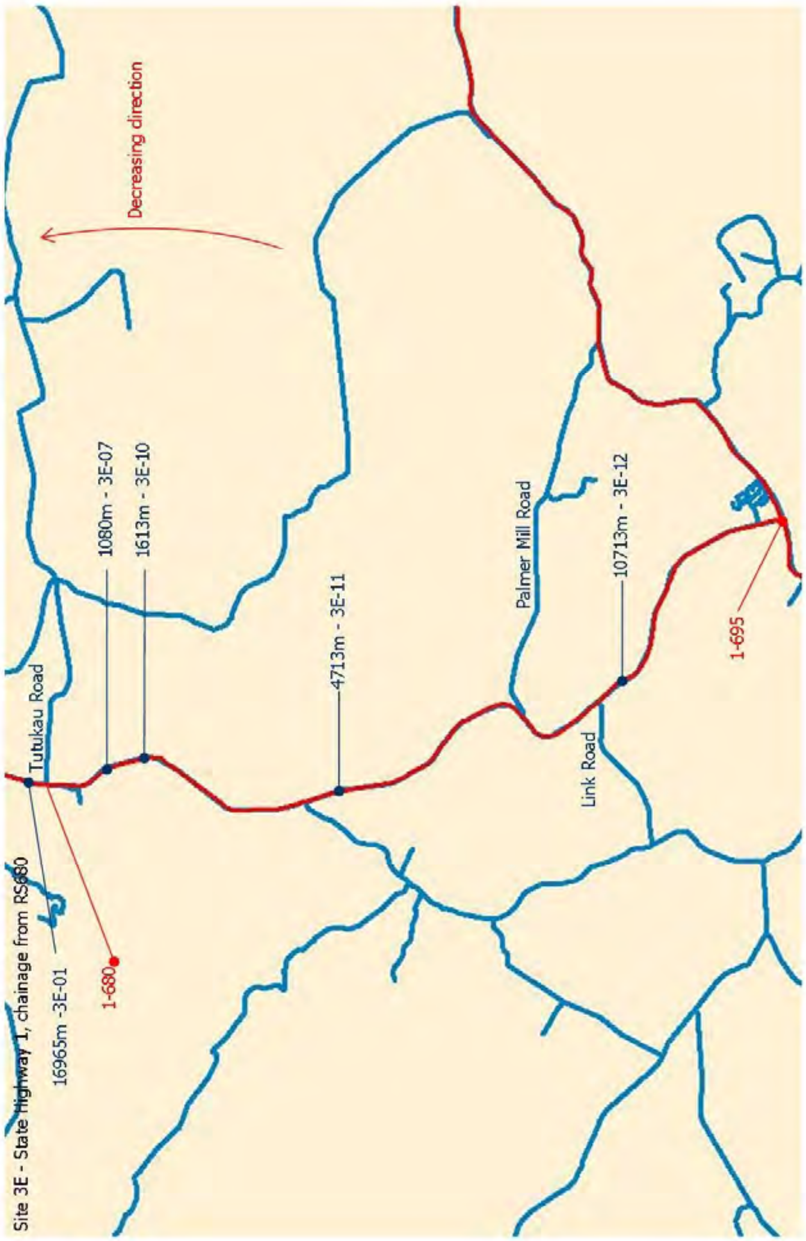
### *A3.1 Characteristics*

Nominal Length of Passing Lane: 557 m
Terrain: Mountainous
Average Gradient of Passing Lane: 5.7%
RAMM AADT: 6511
Measured (3 day average) AADT: 4950
RAMM % HCV: 14%
Measured (3 day average) % HCV: 27%
Monitoring Period: 10/07/2007 to 12/07/2007

### *A3.2 Layout of Automated Classifiers*



### *A3.3 Spatial Location of Automated Classifiers*



### *A3.4 GPS Co-Ordinates of Automated Classifiers*

| Counter Number | Displacement (m) | Latitude | Longitude | Direction A>B | Direction B>A | Comments |
|---|---|---|---|---|---|---|
| 1 | 16695 | S38,31.0780 | E176,02.7804 | South Bound | North Bound | In previous RS (664) |
| 2 | SH1N-RS680/ 297 | S38,31.2734 | E176,02.7685 | South Bound | North Bound | |
| 3 | 587 | S38,31.4320 | E176,02.7706 | South Bound | N/A | Single Lane |
| 4 | 587 | S38,31.4316 | E176,02.7582 | North Bound | South Bound | |
| 5 | 870 | S38,31.5490 | E176,02.8796 | South Bound | N/A | Single Lane |
| 6 | 870 | S38,31.6397 | E176,02.9742 | North Bound | South Bound | |
| 7 | 1080 | S38,31.6397 | E176,02.9742 | South Bound | N/A | Single Lane |
| 8 | 1080 | S38,31.6421 | E176,02.9668 | North Bound | South Bound | |
| 9 | 1300 | S38,31.7442 | E176,03.0199 | South Bound | North Bound | |
| 10 | 1613 | S38,31.9026 | E176,03.0806 | South Bound | North Bound | |
| 11 | 4713 | S38,33.4537 | E176,02.7131 | South Bound | North Bound | |
| 12 | 10713 | S38,36.1027 | E176,04.1449 | South Bound | North Bound | |

### A3.5 Panoramic Views

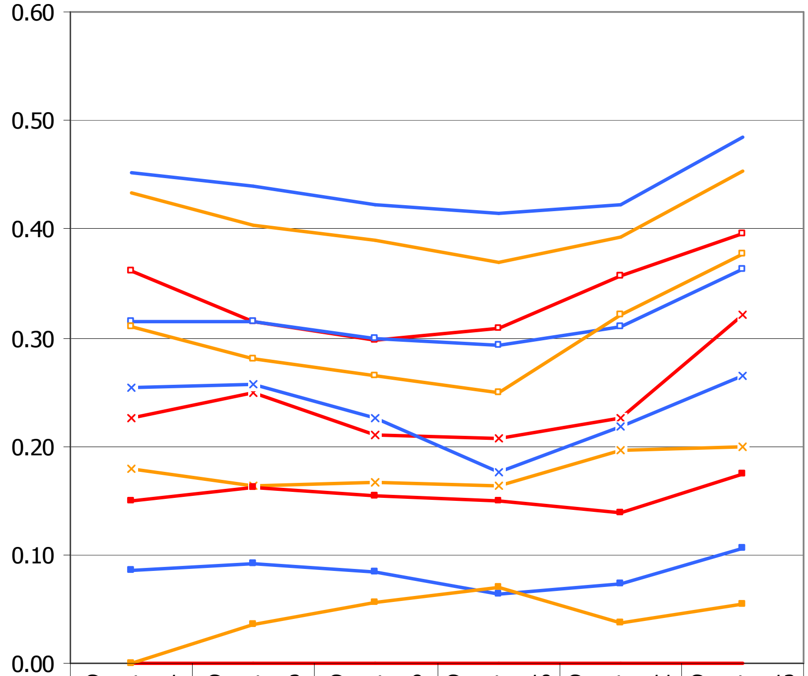
**Start**

**Middle**

**End**

### A3.6 Proportion of vehicles with headway ≤ 4 seconds upstream and downstream of the passing facility at counter locations, treated direction.

**Site 3E:** Proportion with headway <=4.0s (bunched), by vehicles per hour
(Passing lane between Counter 2 and Counter 9)

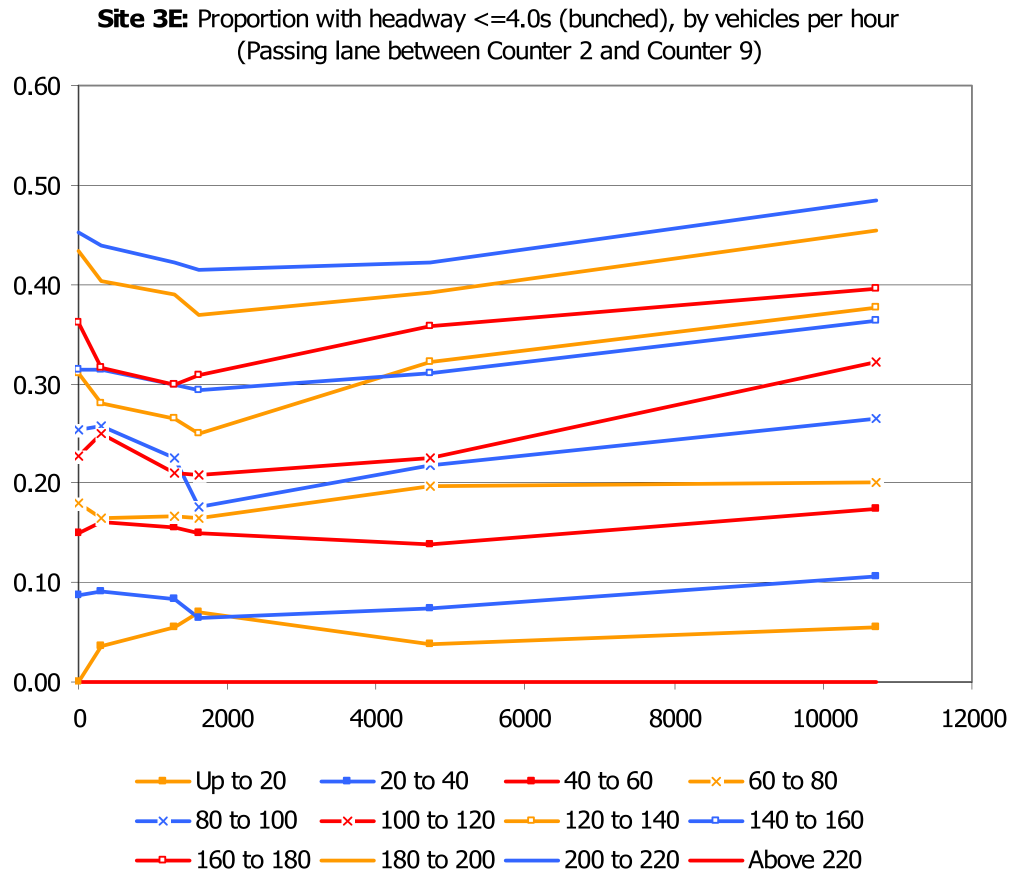



| | Counter 1 | Counter 2 | Counter 9 | Counter 10 | Counter 11 | Counter 12 |
|---|---|---|---|---|---|---|
| Above 220 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| 200 to 220 | 0.4521 | 0.4391 | 0.4218 | 0.4140 | 0.4216 | 0.4844 |
| 180 to 200 | 0.4332 | 0.4036 | 0.3900 | 0.3698 | 0.3921 | 0.4536 |
| 160 to 180 | 0.3615 | 0.3155 | 0.2984 | 0.3079 | 0.3575 | 0.3957 |
| 140 to 160 | 0.3149 | 0.3147 | 0.2997 | 0.2930 | 0.3108 | 0.3630 |
| 120 to 140 | 0.3103 | 0.2808 | 0.2643 | 0.2499 | 0.3210 | 0.3775 |
| 100 to 120 | 0.2262 | 0.2490 | 0.2106 | 0.2077 | 0.2255 | 0.3216 |
| 80 to 100 | 0.2540 | 0.2576 | 0.2253 | 0.1766 | 0.2181 | 0.2647 |
| 60 to 80 | 0.1796 | 0.1638 | 0.1670 | 0.1639 | 0.1970 | 0.1999 |
| 40 to 60 | 0.1499 | 0.1613 | 0.1545 | 0.1489 | 0.1380 | 0.1743 |
| 20 to 40 | 0.0865 | 0.0913 | 0.0835 | 0.0635 | 0.0731 | 0.1056 |
| Up to 20 | 0.0000 | 0.0351 | 0.0556 | 0.0702 | 0.0372 | 0.0541 |

Counter Distance Key:

| Counter No: | RS (km) | Distance from Counter 1 (km) |
|---|---|---|
| 1 | RS664/15.695 | - |
| 2 | RS680/0.297 | 0.602 |
| 9 | 1.300 | 1.605 |
| 10 | 1.613 | 1.918 |
| 11 | 4.713 | 5.018 |
| 12 | 10.713 | 11.018 |

### *A3.7 Variation in proportion of vehicles with headway ≤ 4 seconds with distance, treated direction.*

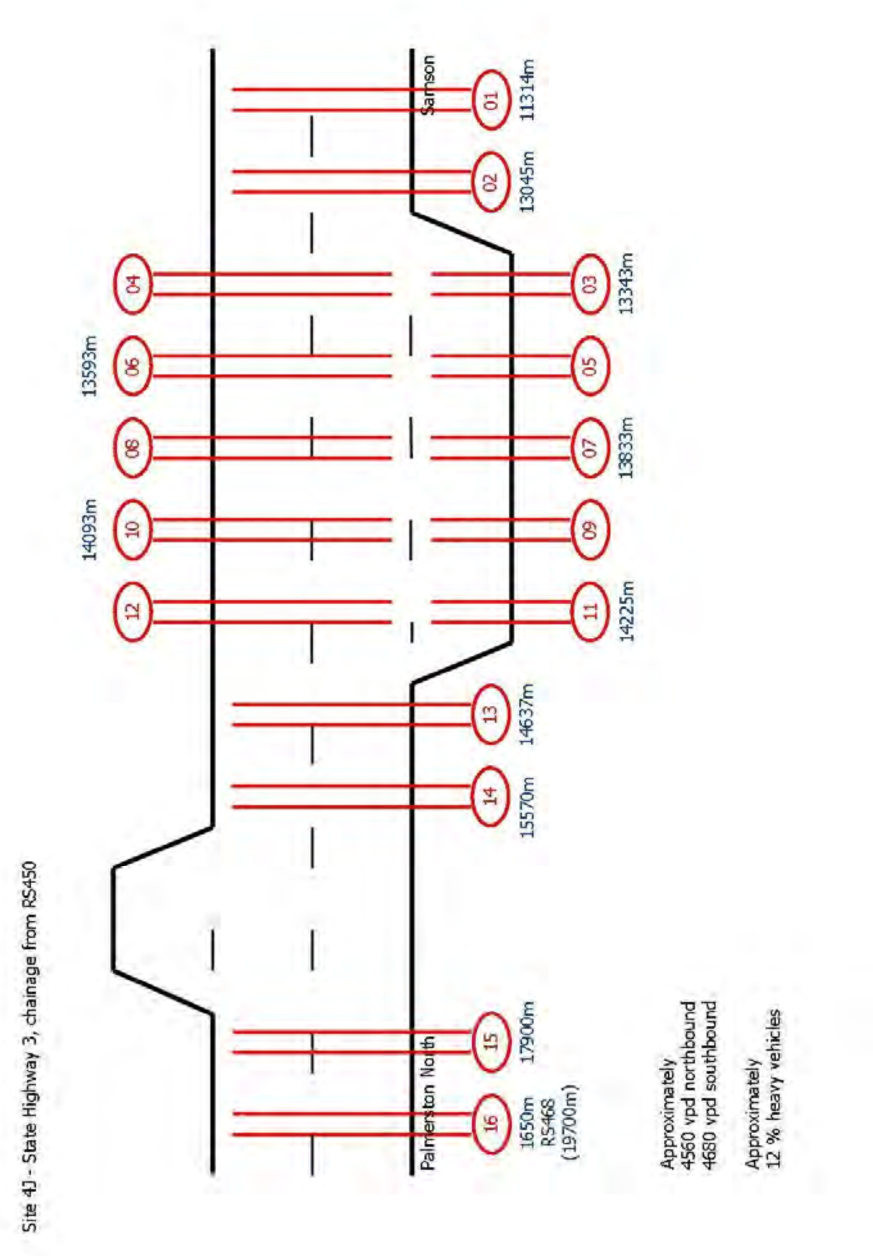


## A4 Long Passing Lane, Site 4j, Increasing Direction

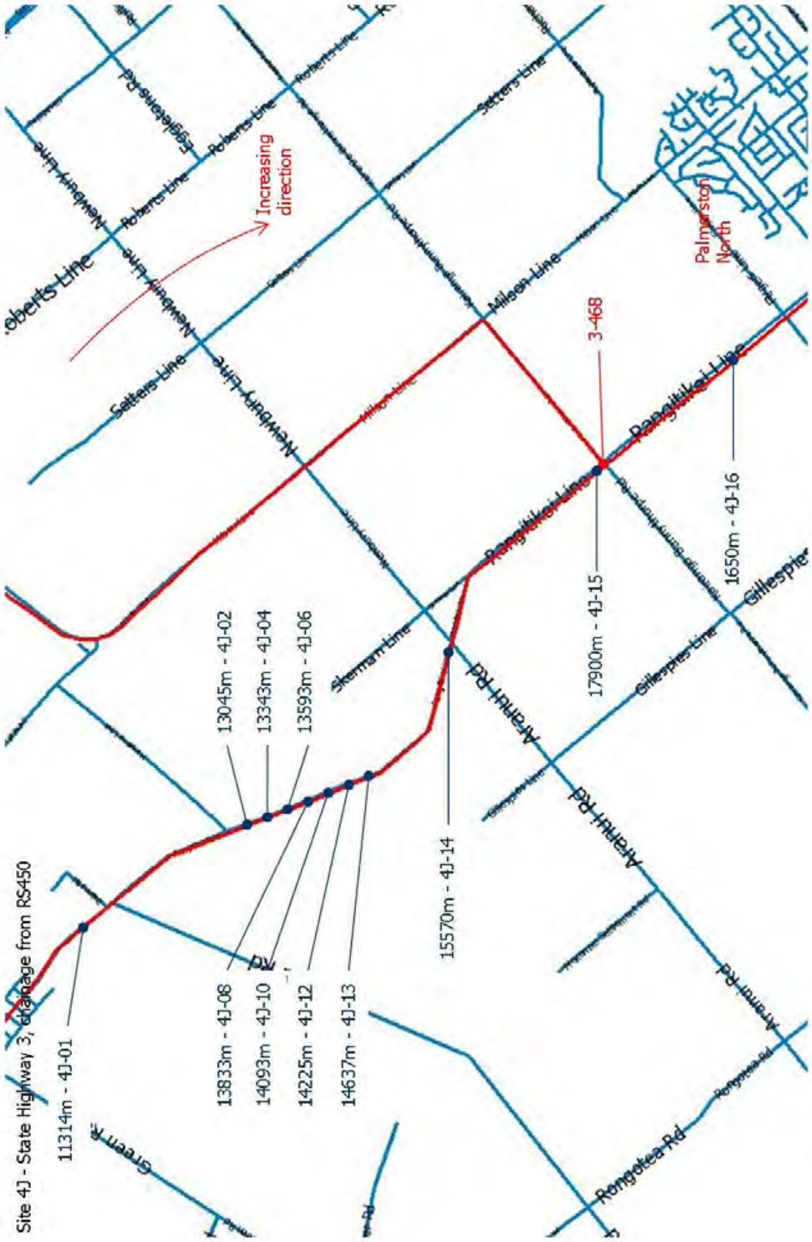
### *A4.1 Characteristics*

Nominal Length of Passing Lane: 936 m
Terrain: Flat
Average Gradient of Passing Lane: 0.4%
RAMM AADT: 8591
Measured (3 day average) AADT: 9240
RAMM % HCV: 11%
Measured (3 day average) % HCV: 12%
Monitoring Period: 18/07/2007 to 20/07/2007

### A4.2 Layout of Automated Classifiers



### *A4.3 Spatial Location of Automated Classifiers*



### A4.4 GPS Co-Ordinates of Automated Classifiers

| Counter Number | Displacement (m) | Latitude | Longitude | Direction A>B | Direction B>A | Comments |
|---|---|---|---|---|---|---|
| 1 | SH3-RS450/ 11314 | S40,16.6321 | E175,31.3778 | South Bound | North Bound | |
| 2 | 13045 | S40,17.3668 | E175,32.1006 | South Bound | North Bound | |
| 3 | 13343 | S40,17.5149 | E175,32.1959 | South Bound | N/A | Single Lane / Sensor Imbalance 28% |
| 4 | 13343 | S40,17.5183 | E175,32.1830 | North Bound | South Bound | |
| 5 | 13593 | S40,17.6359 | E175,32.2623 | South Bound | N/A | Single Lane |
| 6 | 13593 | S40,17.6406 | E175,32.3337 | North Bound | South Bound | Counter vandalised no data |
| 7 | 13833 | S40,17.7586 | E175,32.3337 | South Bound | N/A | Single Lane |
| 8 | 13833 | S40,17.7521 | E175,32.3229 | North Bound | South Bound | |
| 9 | 14093 | S40,17.8881 | E175,32.4064 | South Bound | N/A | Single Lane |
| 10 | 14093 | S40,17.8925 | E175,32.3980 | North Bound | South Bound | |
| 11 | 14225 | S40,17.9541 | E175,32.4453 | South Bound | N/A | Single Lane |
| 12 | 14225 | S40,17.9588 | E175,32.4324 | North Bound | South Bound | |
| 13 | 14637 | S40,18.1550 | E175,32.5601 | South Bound | North Bound | |
| 14 | 15570 | S40,18.4181 | E175,33.0997 | South Bound | North Bound | |
| 15 | 17900 | S40,19.0846 | E175,34.4083 | South Bound | North Bound | |
| 16 | 1650 | S40,19.8189 | E175,35.2445 | South Bound | North Bound | In the next RS (468) would be 19700 |

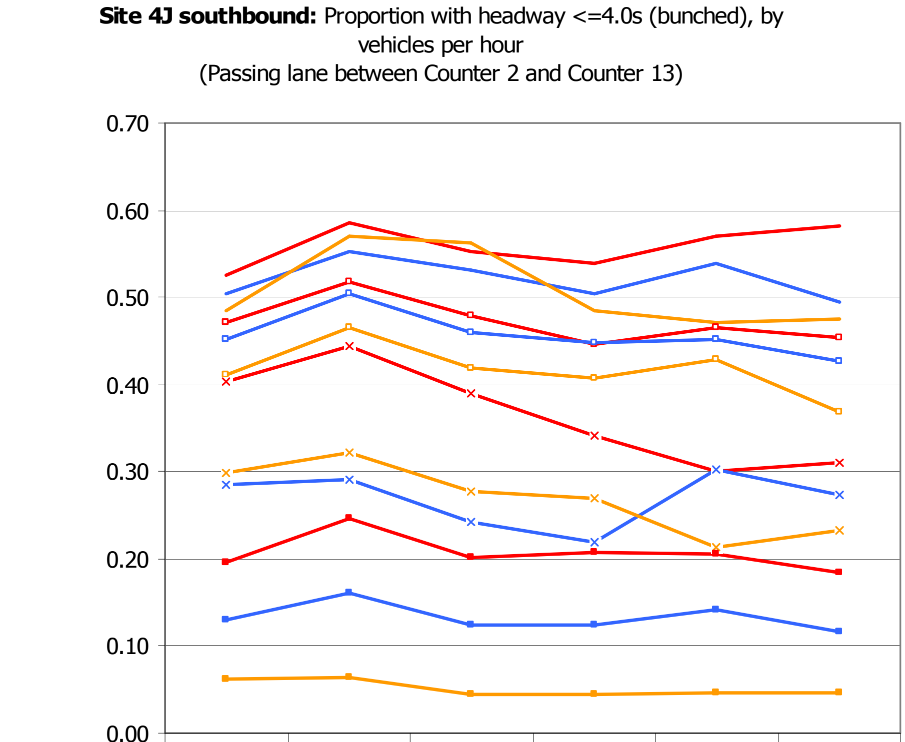
### *A4.5 Panoramic Views (Increasing PL)*


**Start**

**Middle**

**End**

### A4.6 Proportion of vehicles with headway ≤ 4 seconds upstream and downstream of the passing facility (increasing PL) at counter locations, treated direction.

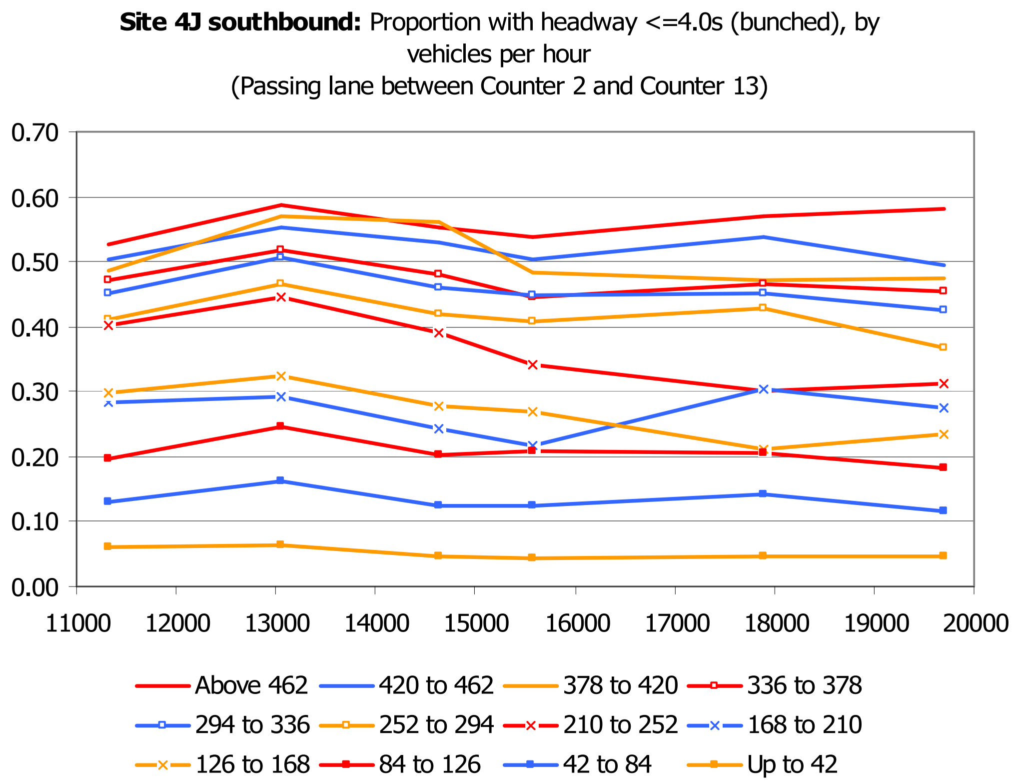



| | Counter 1 | Counter 2 | Counter 13 | Counter 14 | Counter 15 | Counter 16 |
|---|---|---|---|---|---|---|
| Above 462 | 0.5262 | 0.5861 | 0.5521 | 0.5389 | 0.5702 | 0.5817 |
| 420 to 462 | 0.5045 | 0.5518 | 0.5305 | 0.5041 | 0.5383 | 0.4944 |
| 378 to 420 | 0.4854 | 0.5692 | 0.5620 | 0.4839 | 0.4720 | 0.4749 |
| 336 to 378 | 0.4709 | 0.5175 | 0.4791 | 0.4459 | 0.4649 | 0.4542 |
| 294 to 336 | 0.4519 | 0.5049 | 0.4591 | 0.4486 | 0.4527 | 0.4259 |
| 252 to 294 | 0.4114 | 0.4645 | 0.4187 | 0.4065 | 0.4281 | 0.3682 |
| 210 to 252 | 0.4035 | 0.4448 | 0.3894 | 0.3418 | 0.3000 | 0.3111 |
| 168 to 210 | 0.2849 | 0.2908 | 0.2431 | 0.2182 | 0.3026 | 0.2736 |
| 126 to 168 | 0.2979 | 0.3226 | 0.2769 | 0.2701 | 0.2126 | 0.2333 |
| 84 to 126 | 0.1957 | 0.2459 | 0.2025 | 0.2077 | 0.2059 | 0.1834 |
| 42 to 84 | 0.1307 | 0.1610 | 0.1235 | 0.1246 | 0.1414 | 0.1154 |
| Up to 42 | 0.0617 | 0.0643 | 0.0454 | 0.0437 | 0.0469 | 0.0475 |

Counter Distance Key:

| Counter No: | RS (km) | Distance from Counter 1 (km) |
|---|---|---|
| 1 | 11.314 | - |
| 2 | 13.045 | 1.731 |
| 13 | 14.637 | 3.323 |
| 14 | 15.900 | 4.586 |
| 15 | 450/17.900 | 6.586 |
| 16 | 468/1.650 | 8.336 |

### A4.7 Variation in proportion of vehicles with headway ≤ 4 seconds with distance, treated direction - Increasing PL



### A4.8 Panoramic Views (Decreasing PL)

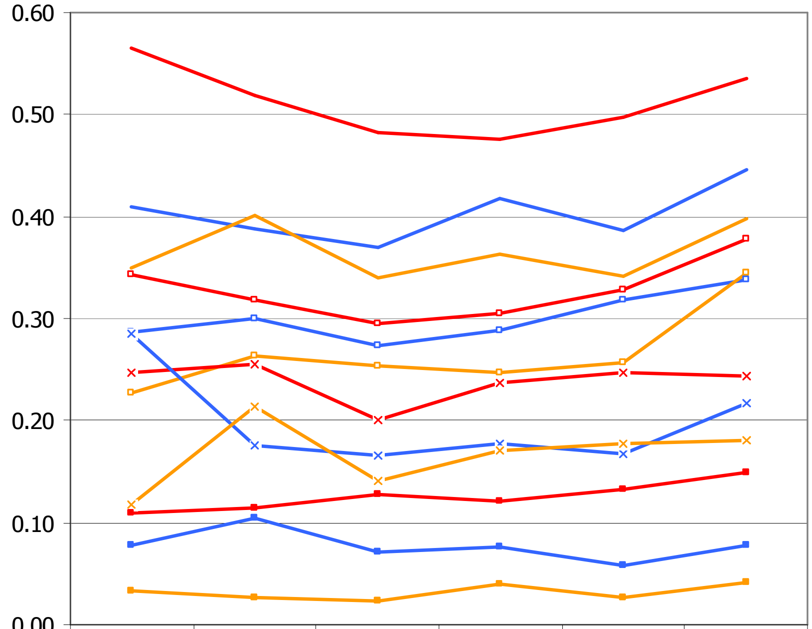
**Start**

**Middle**

**End**

### A4.9 Proportion of vehicles with headway ≤ 4 seconds upstream and downstream of the passing facility (decreasing PL) at counter locations, treated direction.

**Site 4J northbound:** Proportion with headway <=4.0s (bunched), by vehicles per hour
(Passing lane between Counter 15 and Counter 14)

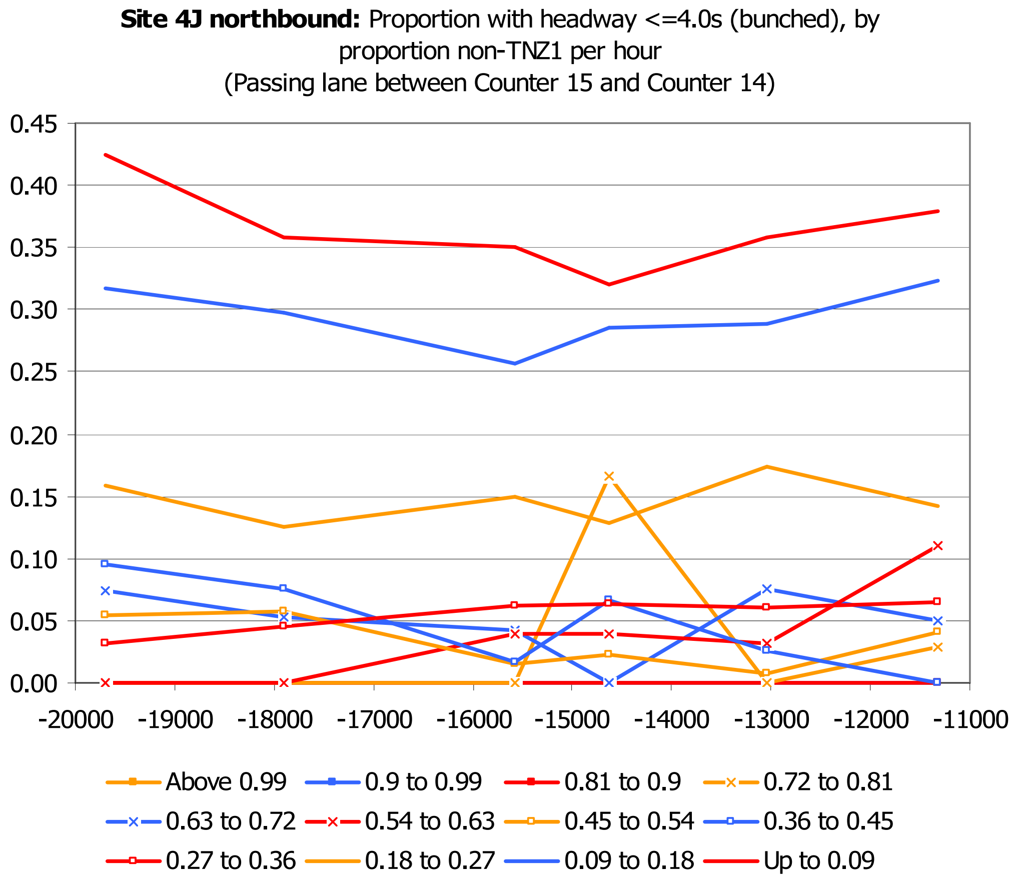



| | Counter 16 | Counter 15 | Counter 14 | Counter 13 | Counter 2 | Counter 1 |
|---|---|---|---|---|---|---|
| Above 330 | 0.5658 | 0.5187 | 0.4816 | 0.4753 | 0.4975 | 0.5347 |
| 300 to 330 | 0.4096 | 0.3877 | 0.3692 | 0.4169 | 0.3864 | 0.4463 |
| 270 to 300 | 0.3492 | 0.4014 | 0.3400 | 0.3638 | 0.3415 | 0.3970 |
| 240 to 270 | 0.3428 | 0.3188 | 0.2948 | 0.3047 | 0.3274 | 0.3781 |
| 210 to 240 | 0.2870 | 0.2992 | 0.2733 | 0.2877 | 0.3184 | 0.3388 |
| 180 to 210 | 0.2272 | 0.2633 | 0.2542 | 0.2468 | 0.2571 | 0.3455 |
| 150 to 180 | 0.2473 | 0.2545 | 0.2003 | 0.2375 | 0.2472 | 0.2442 |
| 120 to 150 | 0.2846 | 0.1752 | 0.1658 | 0.1772 | 0.1669 | 0.2177 |
| 90 to 120 | 0.1180 | 0.2139 | 0.1405 | 0.1703 | 0.1777 | 0.1814 |
| 60 to 90 | 0.1089 | 0.1142 | 0.1277 | 0.1218 | 0.1328 | 0.1487 |
| 30 to 60 | 0.0776 | 0.1041 | 0.0710 | 0.0760 | 0.0587 | 0.0775 |
| Up to 30 | 0.0328 | 0.0267 | 0.0231 | 0.0404 | 0.0262 | 0.0407 |

Counter Distance Key:

| Counter No: | RS (km) | Distance from Counter 1 (km) |
|---|---|---|
| 16 | 468/1.650 | - |
| 15 | 450/17.900 | 1.75 |
| 14 | 15.900 | 3.75 |
| 13 | 14.637 | 5.013 |
| 2 | 13.045 | 6.605 |
| 1 | 11.314 | 8.336 |

### *A4.10 Variation in proportion of vehicles with headway ≤ 4 seconds with distance, treated direction - Decreasing PL*

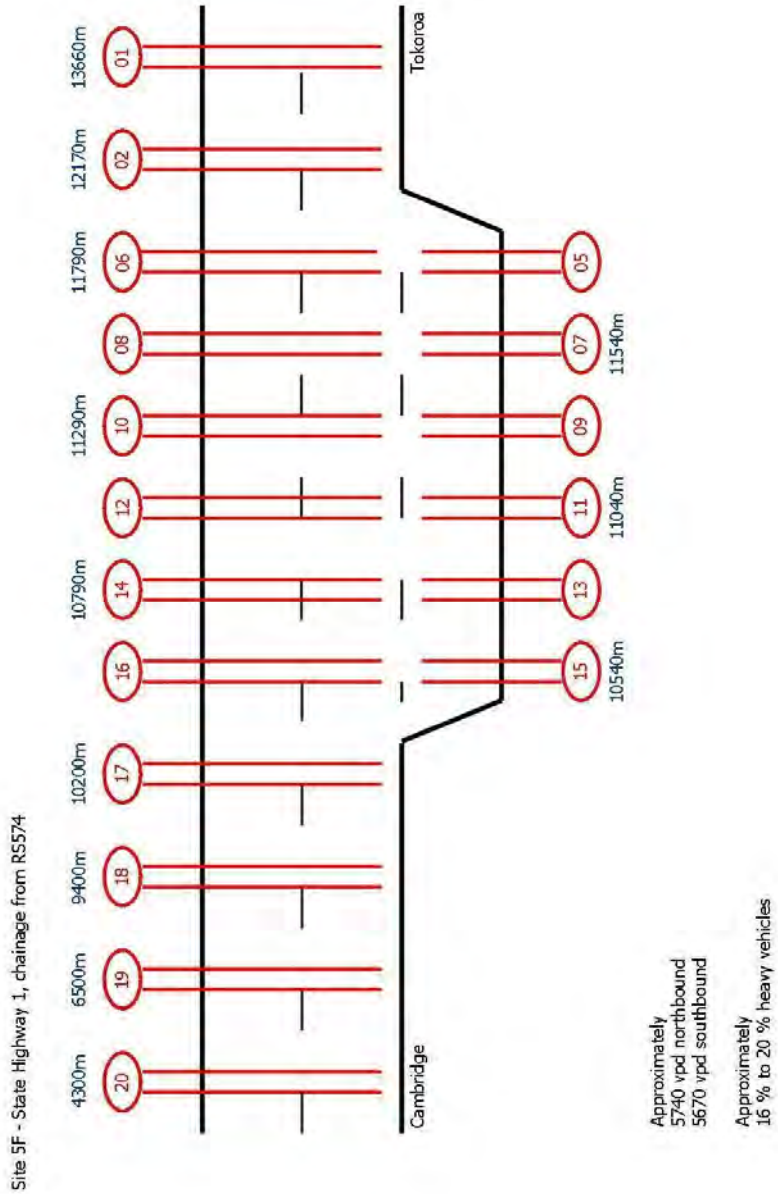


## A5 Long Passing Lane, Site 5f

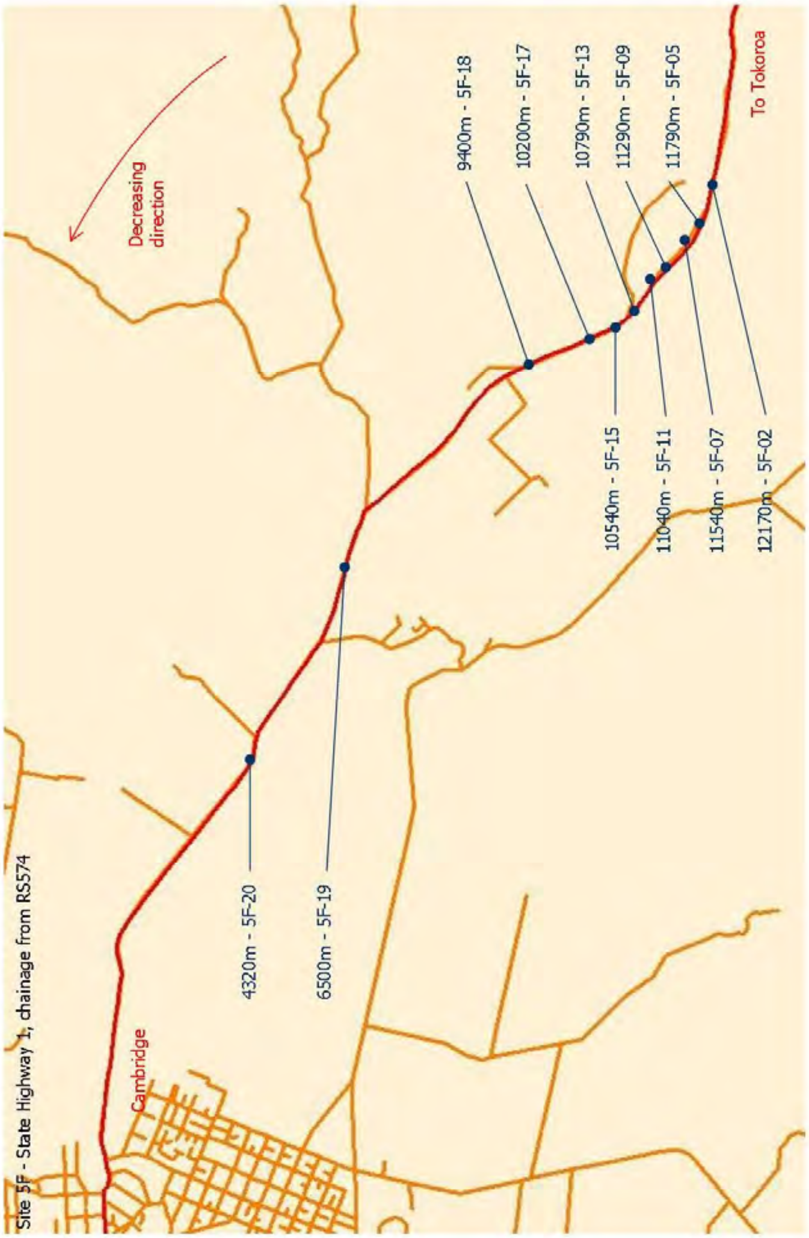
### *A5.1 Characteristics*

Nominal Length of Passing Lane: 1516 m
Terrain: Rolling
Average Gradient of Passing Lane: 0.27%
RAMM AADT: 14368
Measured (3 day average) AADT: 11410
RAMM % HCV: 11%
Measured (3 day average) % HCV: 16-20%
Monitoring Period: 16/07/2007 to 18/07/2007

### *A5.2 Layout of Automated Classifiers*



### *A5.3 Spatial Location of Automated Classifiers*



### *A5.4 GPS Co-Ordinates of Automated Classifiers*

| Counter Number | Displacement (m) | Latitude | Longitude | Direction A>B | Direction B>A | Comments |
|---|---|---|---|---|---|---|
| 1 | SH1N-RS574/ 13660 | S37,56.8670 | E175,36.5808 | South Bound | North Bound | Count data not supplied |
| 2 | 12170 | S37,56.7912 | E175,35.5673 | South Bound | North Bound | |
| 3 | | | | | | Not Set couldn't fit in |
| 4 | | | | | | Not Set couldn't fit in |
| 5 | 11790 | S37,56.7510 | E175,35.3174 | North Bound | N/A | Single Lane |
| 6 | 11790 | S37,56.7416 | E175,35.3198 | South Bound | North Bound | |
| 7 | 11540 | S37,56.6860 | E175,35.1585 | North Bound | N/A | Single Lane |
| 8 | 11540 | S37,56.6791 | E175,35.1644 | South Bound | North Bound | |
| 9 | 11290 | S37,56.5962 | E175,35.0298 | North Bound | N/A | Single Lane |
| 10 | 11290 | S37,56.5887 | E175,35.0386 | South Bound | North Bound | |
| 11 | 11040 | S37,56.5036 | E175,34.9092 | North Bound | N/A | Single Lane |
| 12 | 11040 | S37,56.4980 | E175,34.9177 | South Bound | North Bound | |
| 13 | 10790 | S37,56.4324 | E175,34.7668 | North Bound | N/A | Single Lane |
| 14 | 10790 | S37,56.4215 | E175,34.7737 | South Bound | North Bound | |
| 15 | 10540 | S37,56.3477 | E175,34.6259 | North Bound | N/A | Single Lane |
| 16 | 10540 | S37,56.3364 | E175,34.6361 | South Bound | North Bound | |
| 17 | 10200 | S37,56.2330 | E175,34.5353 | South Bound | North Bound | |
| 18 | 9400 | S37,55.7997 | E175,34.2756 | South Bound | North Bound | Moved forward away from 3 lane (LHS turn) |
| 19 | 6500 | S37,54.8668 | E175,32.7903 | South Bound | North Bound | |
| 20 | 4300 | S37,54.3734 | E175,31.4398 | South Bound | North Bound | GPS Right hand side |

### A5.5 Panoramic Views

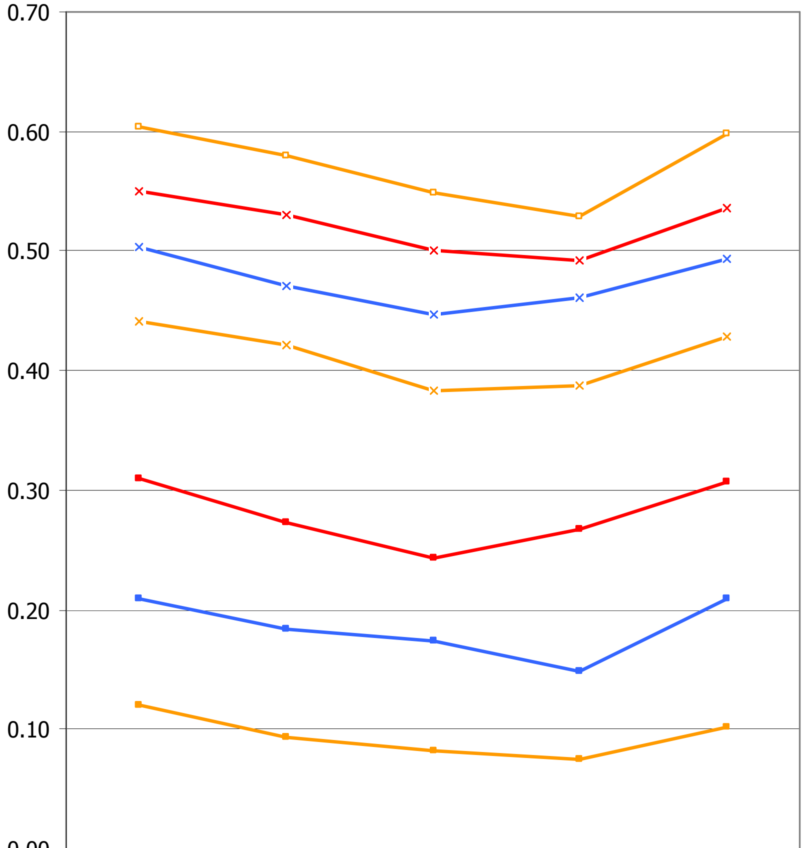
**Start**

**Middle**

**End**

### *A5.6 Proportion of vehicles with headway ≤ 4 seconds upstream and downstream of the passing facility at counter locations, treated direction.*

**Site 5F:** Proportion with headway <=4.0s (bunched), by vehicles per hour
(Passing lane between Counter 2 and Counter 17)

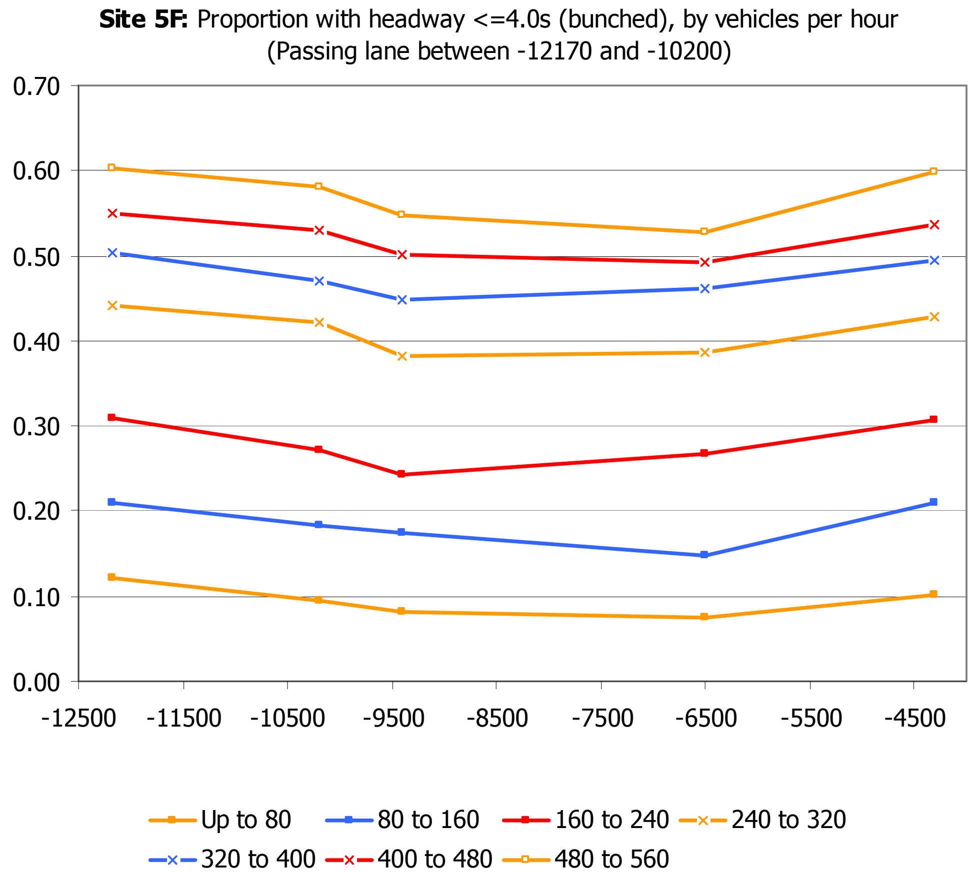



| | Counter 2 | Counter 17 | Counter 18 | Counter 19 | Counter 20 |
|---|---|---|---|---|---|
| 480 to 560 | 0.6033 | 0.5804 | 0.5480 | 0.5283 | 0.5980 |
| 400 to 480 | 0.5508 | 0.5302 | 0.5007 | 0.4922 | 0.5366 |
| 320 to 400 | 0.5040 | 0.4706 | 0.4472 | 0.4605 | 0.4937 |
| 240 to 320 | 0.4413 | 0.4214 | 0.3826 | 0.3870 | 0.4291 |
| 160 to 240 | 0.3091 | 0.2724 | 0.2430 | 0.2679 | 0.3071 |
| 80 to 160 | 0.2094 | 0.1841 | 0.1745 | 0.1481 | 0.2087 |
| Up to 80 | 0.1205 | 0.0940 | 0.0826 | 0.0753 | 0.1011 |

Counter Distance Key:

| Counter No: | RS (km) | Distance from Counter 1 (km) |
|---|---|---|
| 1 | 13.660 | - |
| 2 | 12.170 | 1.49 |
| 17 | 10.200 | 3.46 |
| 18 | 9.400 | 4.26 |
| 19 | 6.500 | 7.16 |
| 20 | 4.300 | 9.36 |

### *A5.7 Variation in proportion of vehicles with headway ≤ 4 seconds with distance, treated direction*

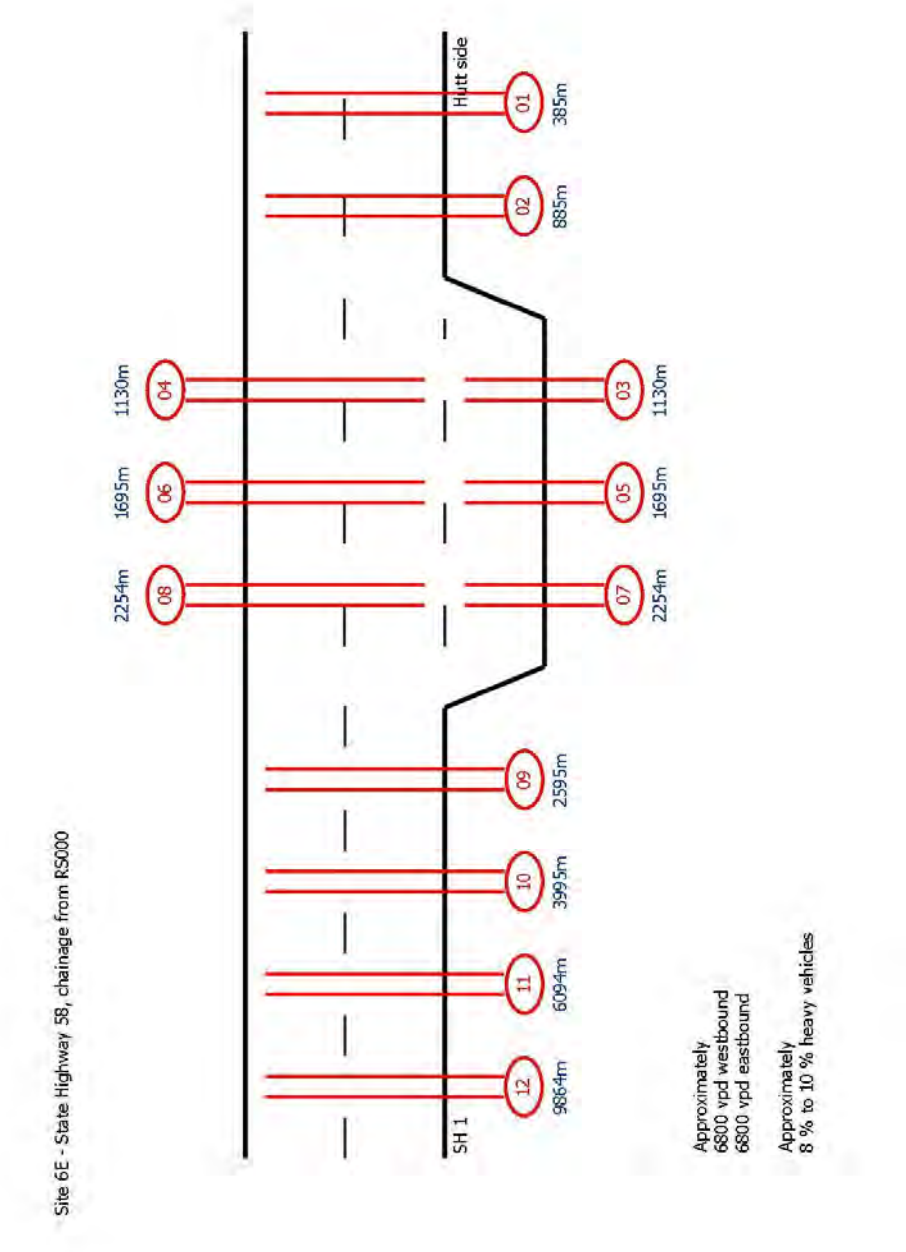


## A6 Long Passing Lane, Site 6e

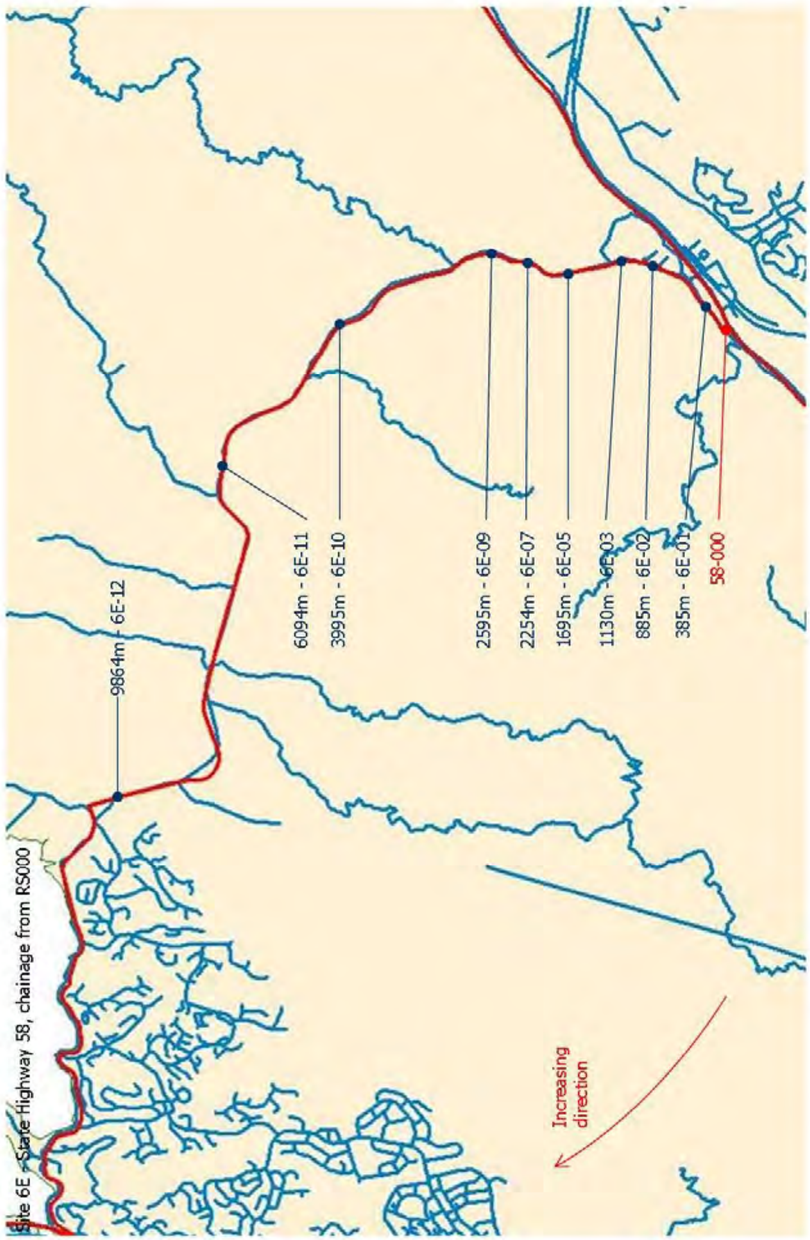
### *A6.1 Characteristics*

Nominal Length of Passing Lane: 1180 m
Terrain: Mountainous
Average Gradient of Passing Lane: 7.2%
RAMM AADT: 13419
Measured (3 day average) AADT: 13600
RAMM % HCV: 3%
Measured (3 day average) % HCV: 8 -10%
Monitoring Period: 24/07/2007 to 26/07/2007

### *A6.2 Layout of Automated Classifiers*



### A6.3 Spatial Location of Automated Classifiers



### *A6.4 GPS Co-Ordinates of Automated Classifiers*

| Counter Number | Displacement (m) | Latitude | Longitude | Direction A>B | Direction B>A | Comments |
|---|---|---|---|---|---|---|
| 1 | SH58-RS0/385 | S41,09.4244 | E174,58.4533 | West bound | East bound | |
| 2 | 885 | S41,09.2323 | E174,58.6847 | West bound | East bound | |
| 3 | 1130 | S41,09.1095 | E174,58.7260 | West bound | N/A | Single lane |
| 4 | 1130 | S41,09.1105 | E174,58.7407 | East bound | West bound | |
| 5 | 1695 | S41,08.8100 | E174,58.6818 | West bound | N/A | Single lane |
| 6 | 1695 | S41,08.8250 | E174,58.6817 | East bound | West bound | |
| 7 | 2254 | S41,08.5374 | E174,58.7161 | West bound | N/A | Single lane |
| 8 | 2254 | S41,08.5244 | E174,58.7281 | West bound | East bound | Traffic was hitting B sensor first (error at put down) |
| 9 | 2595 | S41,08.3600 | E174,58.7800 | West bound | East bound | |
| 10 | 3995 | S41,07.7148 | E174,58.3403 | West bound | East bound | |
| 11 | 6094 | S41,07.0335 | E174,57.3035 | West bound | East bound | |
| 12 | 9864 | S41,06.6019 | E174,55.0920 | West bound | East bound | |

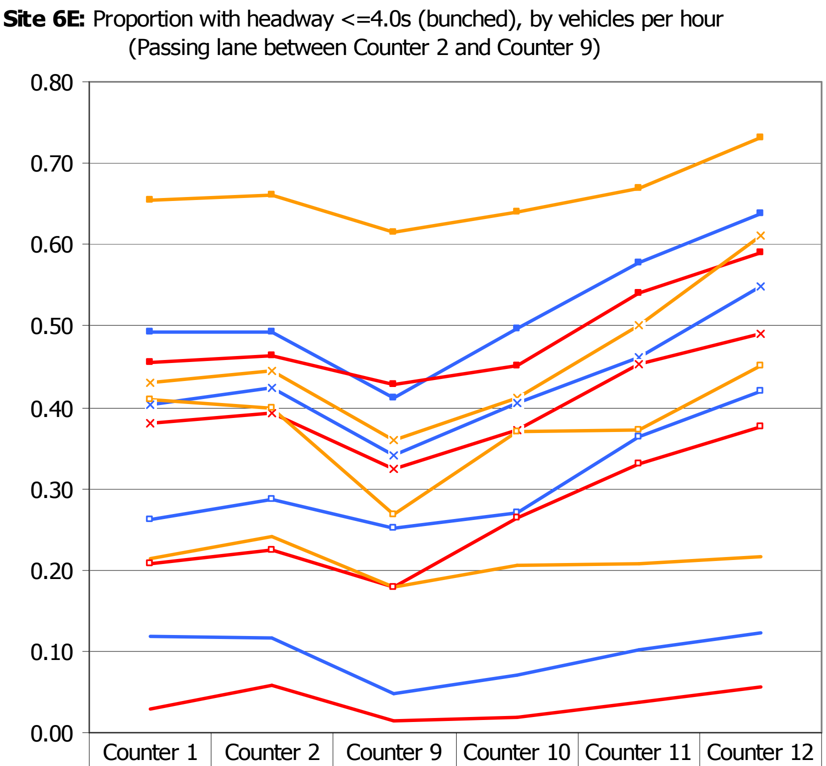
### A6.5 Panoramic Views

**Start**

**Middle**

**End**

### *A6.6 Proportion of vehicles with headway ≤ 4 seconds upstream and downstream of the passing facility at counter locations, treated direction.*

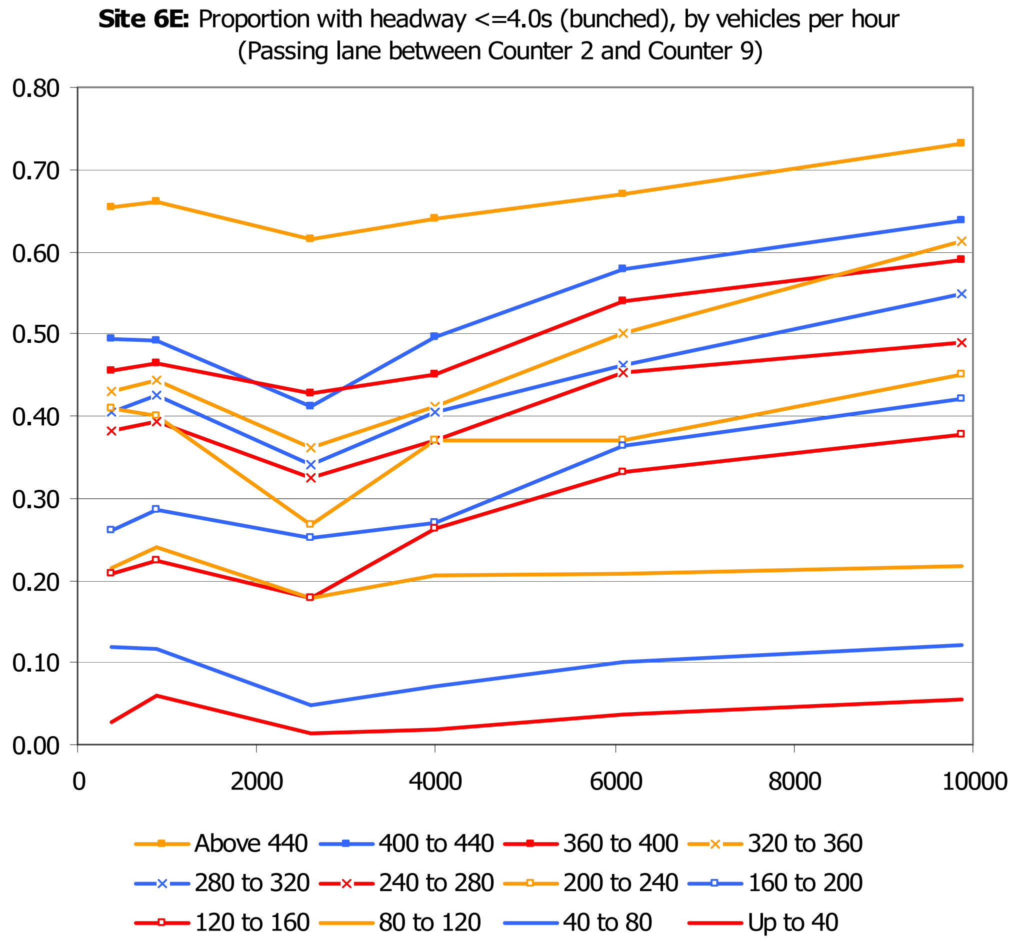



| | Counter 1 | Counter 2 | Counter 9 | Counter 10 | Counter 11 | Counter 12 |
|---|---|---|---|---|---|---|
| Above 440 | 0.6539 | 0.6604 | 0.6152 | 0.6395 | 0.6693 | 0.7314 |
| 400 to 440 | 0.4928 | 0.4917 | 0.4123 | 0.4967 | 0.5778 | 0.6374 |
| 360 to 400 | 0.4555 | 0.4631 | 0.4279 | 0.4511 | 0.5397 | 0.5899 |
| 320 to 360 | 0.4303 | 0.4436 | 0.3603 | 0.4107 | 0.5002 | 0.6115 |
| 280 to 320 | 0.4038 | 0.4243 | 0.3402 | 0.4055 | 0.4609 | 0.5490 |
| 240 to 280 | 0.3807 | 0.3937 | 0.3241 | 0.3711 | 0.4533 | 0.4897 |
| 200 to 240 | 0.4095 | 0.3990 | 0.2673 | 0.3700 | 0.3713 | 0.4512 |
| 160 to 200 | 0.2611 | 0.2866 | 0.2524 | 0.2708 | 0.3637 | 0.4200 |
| 120 to 160 | 0.2082 | 0.2245 | 0.1790 | 0.2638 | 0.3307 | 0.3765 |
| 80 to 120 | 0.2149 | 0.2404 | 0.1778 | 0.2047 | 0.2080 | 0.2168 |
| 40 to 80 | 0.1183 | 0.1168 | 0.0471 | 0.0708 | 0.1015 | 0.1223 |
| Up to 40 | 0.0281 | 0.0586 | 0.0145 | 0.0179 | 0.0371 | 0.0556 |

Counter Distance Key:

| Counter No: | RS (km) | Distance from Counter 1 (km) |
|---|---|---|
| 1 | 0.385 | - |
| 2 | 0.885 | 0.500 |
| 9 | 2.595 | 2.210 |
| 10 | 3.995 | 3.610 |
| 11 | 6.094 | 5.709 |
| 12 | 9.864 | 9.479 |

### A6.7 Variation in proportion of vehicles with headway ≤ 4 seconds with distance, treated direction

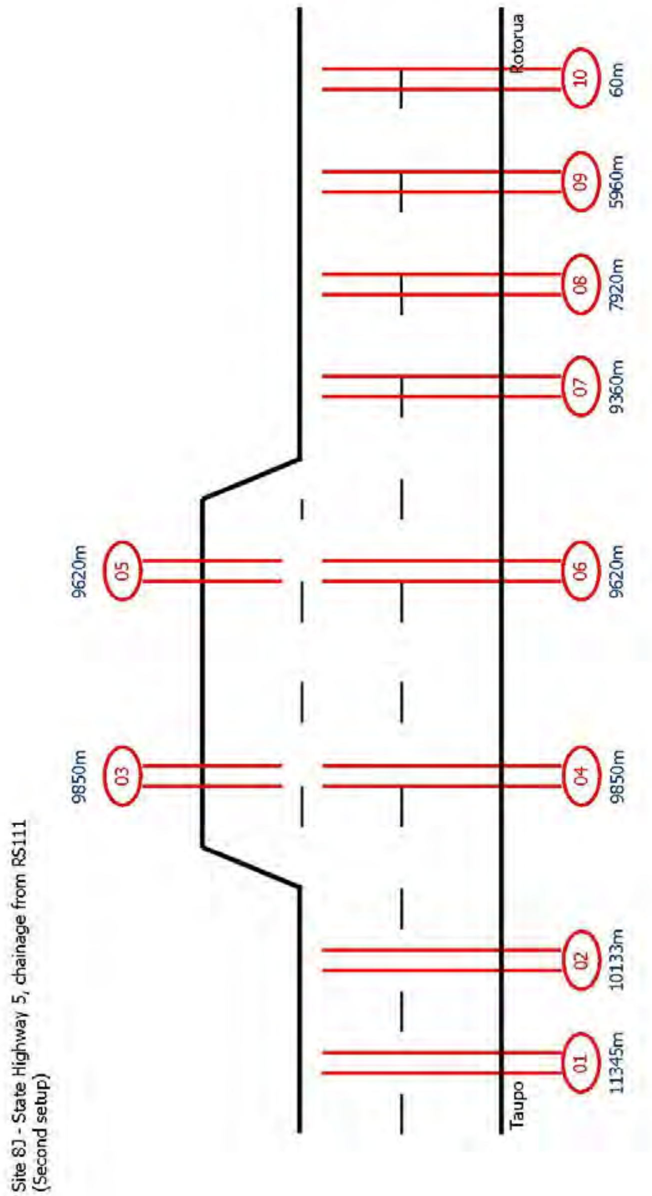


## A7 Slow Vehicle Bay, Site 8j

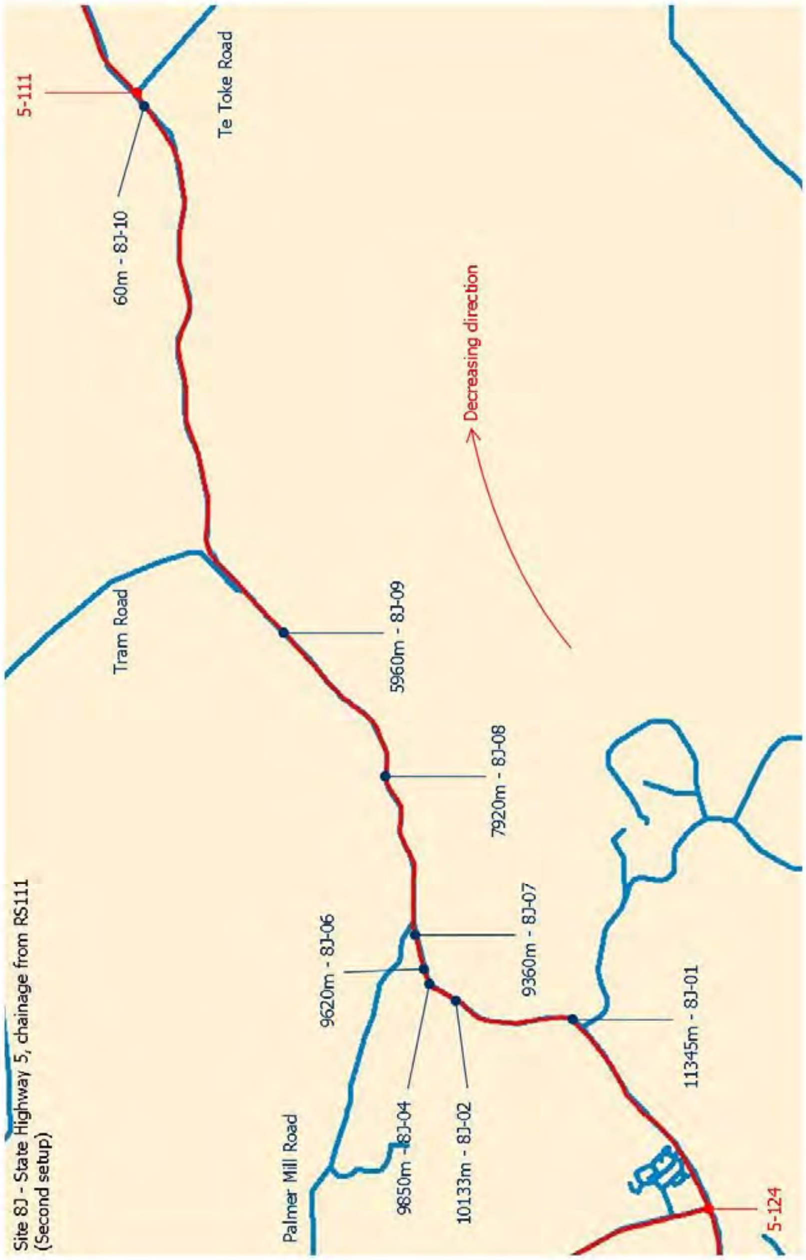
### *A7.1 Characteristics*

Nominal Length of Slow Vehicle Bay: 200 m
Terrain: Mountainous
Average Gradient of Passing Lane: 6.4%
RAMM AADT: 3200
Measured (3 day average) AADT: 3400
RAMM % HCV: 10%
Measured (3 day average) % HCV: 18%
Monitoring Period: 25/07/2007 to 27/07/2007

### *A7.2 Layout of Automated Classifiers*



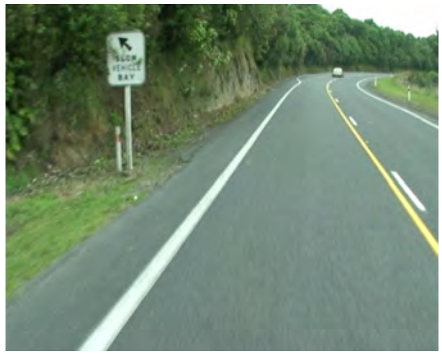
### *A7.3 Spatial Location of Automated Classifiers*



### *A7.4 GPS Co-Ordinates of Automated Classifiers*

| Counter Number | Displacement (m) | Latitude | Longitude | Direction A>B | Direction B>A | Comments |
|---|---|---|---|---|---|---|
| 1 | SH5-RS111/ 11345 | S38,36.6102 | E176,07.2475 | South Bound | North Bound | |
| 2 | 10133 | S38,35.9985 | E176,07.3647 | South Bound | North Bound | |
| 3 | 9850 | S38,35.8689 | E176,07.4618 | North Bound | **N/A** | **Single lane** |
| 4 | 9850 | S38,35.8763 | E176,07.4605 | South Bound | North Bound | |
| 5 | 9620 | S38,35.8220 | E176,07.6108 | North Bound | **N/A** | **Single lane** |
| 6 | 9620 | S38,35.8216 | E176,07.6079 | South Bound | North Bound | |
| 7 | 9360 | S38,35.7937 | E176,07.7868 | South Bound | North Bound | |
| 8 | 7920 | S38,35.6368 | E176,08.7365 | South Bound | North Bound | |
| 9 | 5960 | S38,35.0547 | E176,09.7842 | South Bound | North Bound | |
| 10 | 60 | S38,34.1324 | E176,13.4648 | South Bound | North Bound | |

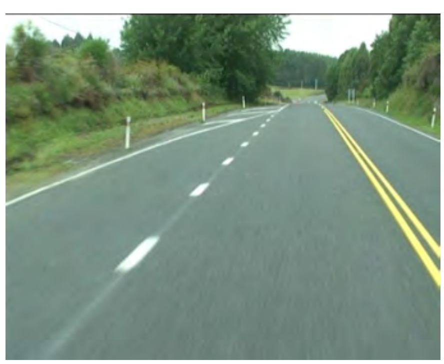
### *A2.5 Panoramic Views*



**Before**

**Start**

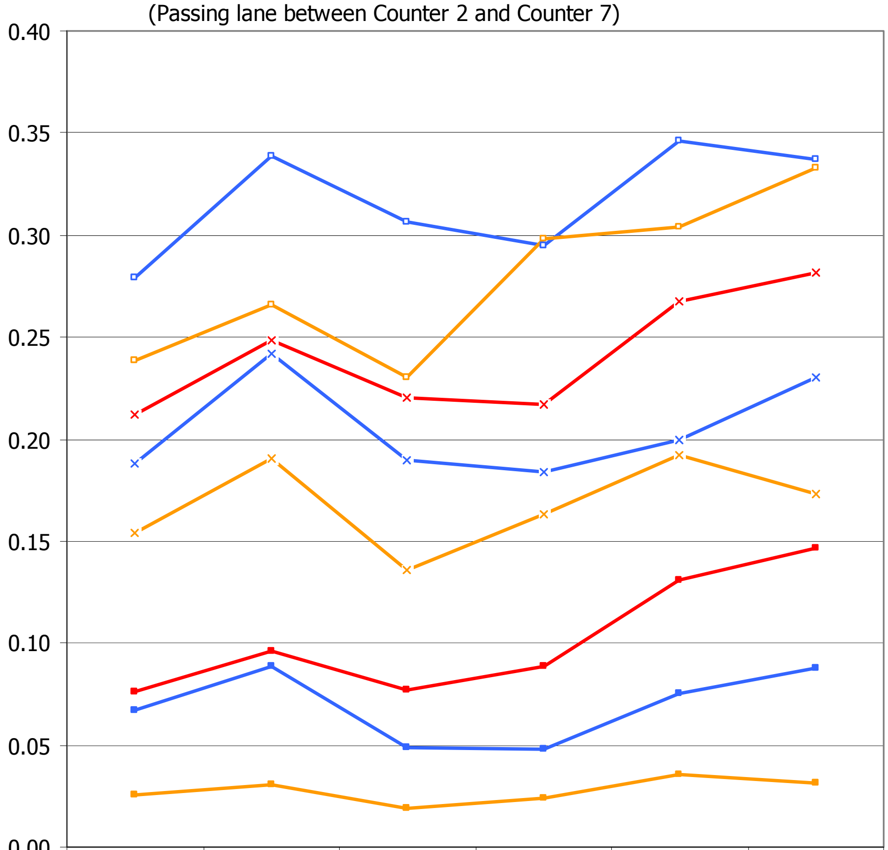
**Middle**



**End**

### *A7.6 Proportion of vehicles with headway ≤ 4 seconds upstream and downstream of the passing facility at counter locations, treated direction.*

**Site 8J2:** Proportion with headway <=4.0s (bunched), by vehicles per hour
(Passing lane between Counter 2 and Counter 7)

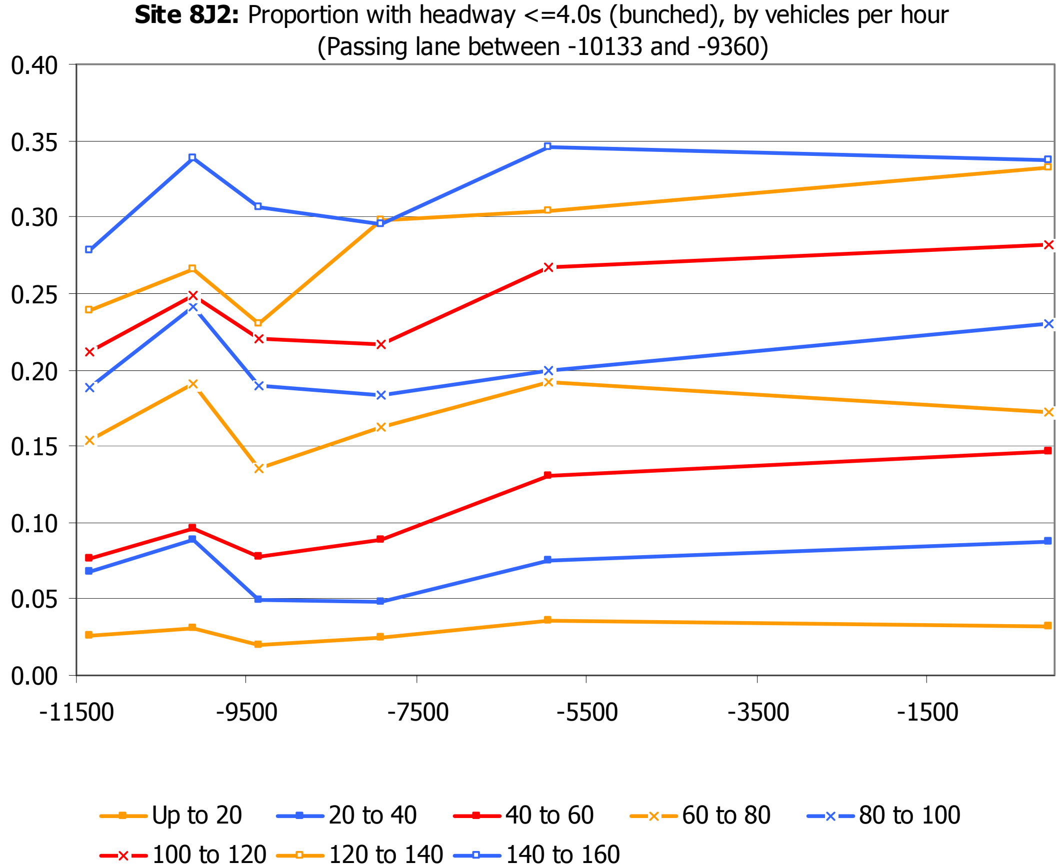



| | Counter 1 | Counter 2 | Counter 7 | Counter 8 | Counter 9 | Counter 10 |
|---|---|---|---|---|---|---|
| 140 to 160 | 0.2787 | 0.3385 | 0.3063 | 0.2949 | 0.3463 | 0.3371 |
| 120 to 140 | 0.2387 | 0.2662 | 0.2302 | 0.2984 | 0.3043 | 0.3326 |
| 100 to 120 | 0.2120 | 0.2486 | 0.2206 | 0.2170 | 0.2674 | 0.2814 |
| 80 to 100 | 0.1883 | 0.2418 | 0.1900 | 0.1836 | 0.1995 | 0.2304 |
| 60 to 80 | 0.1536 | 0.1908 | 0.1358 | 0.1628 | 0.1923 | 0.1729 |
| 40 to 60 | 0.0763 | 0.0958 | 0.0770 | 0.0888 | 0.1306 | 0.1467 |
| 20 to 40 | 0.0673 | 0.0883 | 0.0492 | 0.0476 | 0.0754 | 0.0880 |
| Up to 20 | 0.0256 | 0.0307 | 0.0192 | 0.0243 | 0.0357 | 0.0315 |

Counter Distance Key:

| Counter No: | RS (km) | Distance from Counter 1 (km) |
|---|---|---|
| 1 | 11.345 | - |
| 2 | 10.133 | 1.21 |
| 7 | 9.360 | 1.985 |
| 8 | 7.920 | 3.425 |
| 9 | 5.960 | 5.385 |
| 10 | 0.060 | 11.285 |

### A7.7 Variation in proportion of vehicles with headway ≤ 4 seconds with distance, treated direction



# Appendix B: Theoretical Curve Advisory Speed

## B1 Calculation Procedure

In traditional road design, the "design speed" of a road section is calculated using the equation:

$$V^2 = 127\,R \times (e + f) \qquad \text{(B1)}$$

where
- e = Superelevation (crossfall)
- f = Coefficient of side friction
- V = Design speed (km/h)
- R = Radius (m)

The design speed is, by definition, equal to the 85th percentile speed, and was identified as an important variable in the determination of mean speed on a road section. It is therefore a better measure of vehicle speeds at specific locations than the regulatory speed.

Research has shown that the value of the coefficient of friction (f) used in the design equation is not a factor that governs traffic speed, but rather an outcome of the speed selected by the driver (Bennett 1994). Rawlinson (1983) uses the following relationship:

$$f = 0.30 - 0.0017\,V \qquad \text{(B2)}$$

On this basis, an alternative speed formulation, which is independent of friction was adopted (Wanty *et al* 1995). This is the *Theoretical Curve* ***Advisory Speed*** function (AS) which is defined as:

$$AS = -\left(\frac{107.95}{H}\right) + \sqrt{\left(\frac{107.95}{H}\right)^2 + \left[\frac{127{,}000}{H}\right]\left[0.3 + \frac{X}{100}\right]} \qquad \text{(B3)}$$

where
- AS = Theoretical Advisory Speed (km/h)
- X = % Crossfall (sign relative to curvature)
- H = Absolute Curvature (radians/km) = (1000m / R)

Using equation (C3), the road geometry data in the NZ Transport Agency's RAMM database can be used to generate a speed measure over the state highway network. This equation tends to give very high values for speed on straight sections so, in data analysis, speeds are capped to, say, 100 km/h in rural areas. Automatic calculations of the formula also have to account for sections of road with H=0, by assigning an arbitrarily large radius to these sections (e.g. 99999).

Gradient effects can also be incorporated into the AS by using a simple formula to limit speeds (derived from Bennett 1994):

$$AS \leq 125 - G \times 5 \qquad \text{(B4)}$$

where G = % Gradient (positive = uphill)

This helps to dampen speeds on straight uphill slopes. For example, on an 8% uphill grade, the AS cannot exceed 85 km/h. The above formula applies to car speeds; for truck speeds, a different calculation is needed and applied to steep downhill grades as well.

Using this basic calculation, “speed profiles” can be produced from road geometry data over a length of highway.

## B2 References

Bennett, C.R. 1994. A Speed Prediction Model for Rural Two-lane Highways. *School of Engineering Report 541*, University of Auckland.

Rawlinson, W.R. 1983. The ARRB Road Geometry Instrumented Vehicle - General Description. *ARRB Internal Report AIR 276-2*, ARRB, Vermont South, Victoria, Australia.

Wanty, D.K., McLarin, M.W., Davies, R.B., Cenek, P.D. 1995. Application of the road geometry data acquisition system (RGDAS). *7th World Conference on Transport Research*, Sydney, Australia. (not published in the proceedings)