ATTACHMENT D
COMPARISON WITH INTERNATIONAL PRACTICE

Comparisons for PLs

British Columbia guidelines suggest a range of passing lane lengths with the PO Policy long-term framework being at the lower end of their PL length range (Ministry of Transportation & Highways, British Columbia, 1998).

These length ranges are based on US research into the most cost effective length of passing lane relative to traffic flow rate (Harwood, Hoban & Warren, 1988).

Kansas and Alberta, which are located in predominantly flat terrain, prefer longer PL lengths of about 2 km or more (Mutabazi, Russell & Stokes, 1999 and Alberta Infrastructure, 1999).

PL lengths greater than 1.5 km were less effective outside of peak hours, unless there are high flows throughout most of the day (Harwood & Hoban, 1987).

A comparison between Australian and Canadian guidelines showed that shorter PL lengths in Australia were used as they were applied progressively to the network as traffic volumes increased (Hoban & Morrall, 1986).

Whereas, longer PLs in Canada were a result of not developing the network until larger AADTs, which meant that initial PLs had to cope with higher demands from both large volumes and longer spacings. The stricter Canadian policy for no-overtaking lines was also a factor on longer Canadian passing lane lengths (Hoban & Morrall, 1986).

For German highway cross-sections, the upper limit for 2+1 lanes is about 20,000-25,000 vpd (Brilon & Weiser, 1995).

Research into NZ Passing Lanes and SVBs

Table D1 compares NZ research (Cenek & Lester, 2008) with various points on the Policy's long-term framework. Generally, there is close alignment between the Policy framework and the research.

The surveyed results are on top and not bracketed. Policy values are underneath and bracketed.

Except for sites 4j and 6e, the surveyed projected AADT interval for most sites is based on 55%/45% directional split and peak hour flow 10.5 % of AADT (Approx 125th percentile highest hour).

For site 4j survey results, a 65%/35% directional split and peak hour flow 12 % of AADT was used. For site 6e survey results, a 55/45% directional split & peak hour flow 12 % of AADT was used. Sites 4j & 6e are on rural commuter routes. Site 4j was later found to be a regular location for speed enforcement using mobile speed cameras and this speed enforcement would explain the site's relative under-performance.

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Table D1. Comparison between NZ Surveyed & Policy Layouts (2 second headway)

Site Details			Flow Characteristics		Layout	
Site	**PL/SVB Gradient (%)**	**D/stream Gradient (%)**	**LVT & HCV (%)**	**Projected AADT Interval (vpd)**	**PL or SVB Length (km)**	**Downstream Operational Length (km)**
2e	6.8 [1] (0-3)	0-3 (0-3)	8-13 (< 20)	4,500-5,700 (4,000-5000) (5,000-7,000)	0.6 (0.6-0.8) (1.2)	6.8 (10) (5 or 10) [2]
3e	5.7 (> 6)	> 6 (> 6)	17-21 (< 20)	3,300-4,300 (2,000-4,000)	0.6 (0.6-0.8)	10 (10)
4j	0.4 (0 – 3)	0 – 3 (0 – 3)	5-10 (< 20)	4,400-6,200 (4,000-5,000) (5,000-7,000)	0.9 (0.6-0.8) (1.2)	8.3 [3] (10) (10)
5f	0.3 (3 – 6)	3 – 6 (3 – 6)	13-20 (< 20)	6,100-9,700 (7,000-10,000)	1.4 (1.5)	6.9 (5 or 10)
6e	7.2 (> 6)	> 6 (> 6)	7-13 (< 20)	10,500-12,200 (10,000-25,000)	1.2 (1.2-1.5)	3.9 (5)
8j	6.4 (> 6)	> 6 (> 6)	14-15 12-18 (< 20)	2,900-3,300 2,000-2,500 (2,000-4,000)	0.33 0.33 (0.6-0.8)	2.4 8 (10)

Note: 1. On localised gradient (6.8%) steeper than range for flat (0-3 %) road gradient.
2. For projected 5,000-7,000 vpd, Policy has 5 or 10 km spacing but generally at the lower end of projected AADT interval use 10 km spacing.
3. Regression analysis based on extrapolation from counters approx 1.2 & 3.5 km downstream gave 13.5 km. From mathematical model, calculated 8.3 km operational length.

Comparisons for OT Treatments

Alberta provides double yellow lines at an AADT of 4,000 vpd (Morrall & Hoban, 1985). At low flows, there was about 10 % illegal overtaking on sections of road with adequate sight distance (Morrall, Werner & Kilburn, 1986).

However, for one surveyed section of Canadian state highway, the ADT varied between a low of about 2,300 vpd for winter and a high of about 9,800 vpd during summer. The highest daily peak traffic flow was about 12,800 vpd (Morrall & Blight, 1985). These varied flows averaged about 4,700 vpd. During public holiday weekends, there were upwards of 25 % RVs in the traffic stream.

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Comparisons for OT Treatments continued

NZ does not usually have such wide variations in AADT with high RVs. Therefore, Transit would not use 4,000 vpd, as a threshold for double yellow line markings.

A study of US passing lane sites (Harwood, St John and Warren, 1985) showed that there was no adverse safety issues associated with overtaking using the opposing lane, up to 400 vph (one-way). One-way flows above 400 vph may still be safe but there was not enough data to statistically confirm any safety issues.

Within the PO Policy's long-term framework, this approximates to an AADT of about 7,600 vpd (i.e. (400 vph/0.55) x 10.5 = 7,636 vpd), which is close to the projected AADT 7,000 vpd interval. The 10.5% value relates to the 125th percentile hour on NZ non-recreational state highways (Land Transport NZ, 2006).

Beyond 600 - 700 vph one-way, there was a reduction in overtaking with no overtaking after 1,000 vph one-way (Harwood & Hoban 1987). Within these Guidelines, the 600 - 700 vph one-way flow approximates to AADTs of 11,500 - 13,400 vpd (55% / 45% directional split and peak hour flow of 10.5% AADT).

Transit will consider more restrictive centreline treatments (i.e. central median cables, gap separation) after about 10,000 – 12,000 vph two-way, particularly if one-way flows are consistently high throughout the day. Restrictive centreline treatments may be applied at lower AADTs depending on one-way flows, crash history and experience with similar situations.

References

The following publications are referred to within this attachment:

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11. Morrall J.F. & Hoban C.J., 1985. "Design Guidelines for Overtaking Lanes", Traffic Engineering & Control 26 (10), Oct. 1985, pp.58-69.

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