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## CHAPTER 900 AUXILIARY FACILITIES

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## 910 SLOW MOVING VEHICLE PULLOUT

### 910.01 INTRODUCTION

Slow Moving Vehicle Pullouts are primarily for “older” 2-lane highways where passing opportunities are limited and where slow moving vehicles impact the Level of Service and cause unacceptable platooning. These are predominantly summer recreational routes through areas where the cost of conventional passing or climbing lanes would be prohibitive, relative to the benefits. Some jurisdictions call these Turnouts.

### 910.02 GENERAL

When choosing a pullout location, you should balance the passing opportunities for each direction and avoid long no-passing sections. Signing and pavement marking are in accordance with the Ministry’s Manual of Standard Traffic Signs & Pavement Marking<sup>1</sup>. Avoid Pullouts on downhill sections.

On long winding sections of roadway, locate the pullouts so as to reduce the length of the continuous “No Passing” zones to 15 km or less in mountainous terrain and 10 km or less in level or rolling terrain. Large trucks tend to avoid pullouts, especially on a grade. Pullouts should not be mixed with passing or climbing lanes. No accesses are permitted within pullouts and they should also be avoided opposite the pullout.

Pullouts may also be considered on long uphill grades when a truck climbing lane cannot be built and where speed reductions of at least 20 km/h below the posted or 85th percentile speed are encountered. Refer to TAC Geometric Design Guide for Canadian Roads **Figure 2.1.8.3** for heavy trucks’ performance curves on grades.

### 910.03 GUIDELINES FOR INSTALLATION

Pullouts should be considered when Level of Service B cannot be maintained due to the presence of slow moving vehicles and insufficient passing opportunities. According to the Highway Capacity Manual (Transportation Research Board, HCM2000 metric edition, Chapter 20), at the Level of Service B/C interface the percent time spent following is 50% on Class I highways, for which efficient mobility is paramount.

When the percentage of no passing zones exceeds 60% and the accident history or field observations indicate that there is an excessive amount of dangerous passing manoeuvres due to driver frustration, pullouts should be considered even though Level of Service B is not exceeded during peak hours. (The peak hour in rural situations can be interpreted to be a summer mid-day hour, typically about the 100<sup>th</sup> highest hourly volume of the year).

Table 910.A gives hourly directional volumes ( $V_{APP}$ ), bi-directional Average Annual Daily Traffic (AADT) and Summer Average Daily Traffic (SADT) with the corresponding distance over which it is likely that queues (delayed vehicles) exceeding 5 vehicles (platoon of 6 vehicles including the slow vehicle) would develop. This represents the spacing of Pullouts. Intermediate values can be interpolated.

Table 910.A is derived from a Queue catch-up model provided by ADI Ltd<sup>2</sup>., using the following assumptions:

- The directional volume is the peak hourly flow rate in a 60/40 traffic split.
- There are 20% slow moving vehicles (at 20 to 10 km/h below the desired speed of 80 km/h).
- The peak hour is 15% of the AADT.
- The SADT is 1.5 times the AADT.

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**Table 910.A Pullout Spacing Recommendations**

$V_{APP}$	Pullout Spacing (km) (20/10 km/h below desired speed)	SADT	AADT
20	30/50	400	250
40	15/25	700	450
60	10/17.5	1050	700
80	7.5/12.5	1350	900
100	6/10	1750	1150

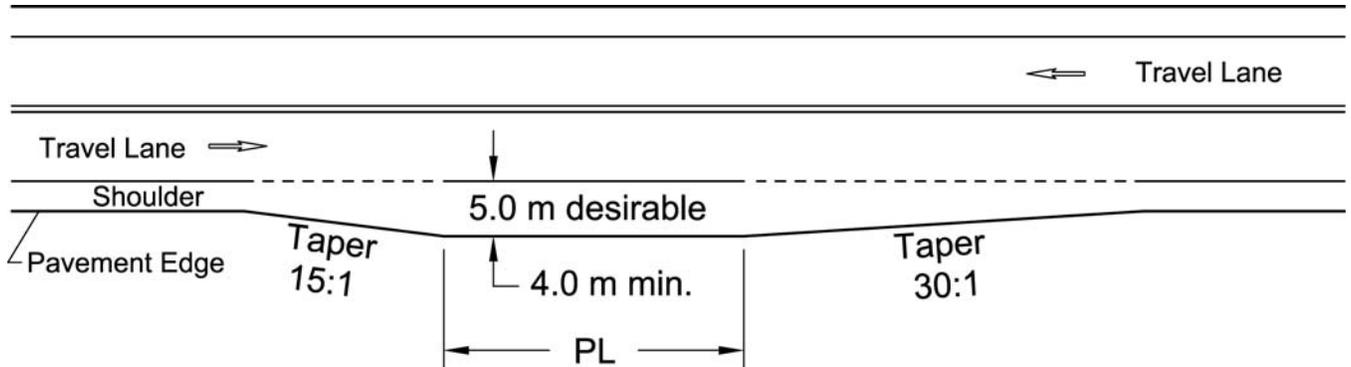
## 910.04 DESIGN ASSUMPTIONS

The following design assumptions were used to obtain the dimensions listed in Table 910.B - Pullout Lengths.

- Reference Speed: posted speed or 85<sup>th</sup> percentile speed, whichever is greater;
- Slow moving vehicles are going 20 km/h less than the reference speed;
- Minimum PL: The slow moving vehicle (SMV) brakes safely to a stop within the PL;
- Desirable PL: The SMV reduces speed to 35 km/h less than the reference speed through the start taper;
- Maximum PL: This is the limit above which the pullout becomes a passing lane. The assumption for the maximum length is that the SMV continues at 20 km/h below the reference speed as they drive through the pullout and let 5 vehicles go by. If there are more than 5 vehicles passing, then the SMV's will have to slow down and come to a stop or merge when safe to do so;
- No Access within the length of the pullout or opposite the pullout;
- Speed of all vehicles other than SMV's is the reference speed. There are 5 passing vehicles;
- Stopping Sight Distance for the reference speed should be available through the entire length.

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**Figure 910.A Typical Slow Moving Vehicle Pullout Configuration**



Note: This is a shoulder widening. The minimum 4.0 m width is to avoid pavement degradation by off-tracking or wide vehicles. Pavement design should be as per travel lanes.

**Table 910.B Pullout Lengths**

Reference Speed km/h	Minimum PL (m)	Desirable PL (m)	Maximum PL (m)
50	30	70	200
60	45	120	300
70	65	190	500
80	85	270	600

Note: Parking should be prohibited in the pullout area.

**References:**

1. Manual of Standard Traffic Signs & Pavement Markings, BC Ministry of Transportation, Engineering Branch.
2. Lyall, Peter D. and Jagannathan, R “Auxiliary Lane Warrants for Two-Lane Highways, Volume I: Report”, ADI Limited, Victoria, BC. 1993.

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## 920

# TRUCK CLIMBING LANE WARRANTS AND DESIGN

### 920.01 INTRODUCTION

Climbing lanes are introduced on steep upgrades to provide a lane for trucks and other slow moving vehicles whose speed drop because of the grade. Climbing lanes are warranted by specific grade effects on Level of Service and/or operating speed, rather than a lack of passing opportunity over a long stretch of a two lane highway. Climbing lanes may be added along with passing lanes as part of a corridor upgrade to improve the level of service by breaking up vehicle platoons. This is a planning exercise involving an operation analysis of long section of highway. When the technical warrants have been met the general procedure should be to: 1) determine the optimum planning and design parameters to fine-tune the location, start and end of the climbing lane; 2) estimate the costs of providing the climbing lane; and 3) do a benefit-cost analysis. For more information on the planning of passing and climbing lanes, refer to the ADI report<sup>1</sup>. For information on the design and co-ordination of passing lanes with climbing lanes refer to Section 930.

### 920.02 WARRANT

A climbing lane is generally recommended if all three of the following criteria are satisfied:

- A speed reduction of 15 km/h for a 180 g/w truck (300 lb/hp);
- Upgrade traffic flow exceeds 200 veh/h; and
- Upgrade truck traffic exceeds 20 veh/h.

Information with regard to truck loading and haul direction will influence the decision to provide climbing lanes. The need for climbing lanes is reduced if truck traffic in the upgrade direction is predominantly empty backhaul.

### 920.03 DESIGN GUIDELINES

Use the TAC Geometric Design Guide for Canadian Roads, Figure 2.1.8.3 – Performance Curves for Heavy Trucks, 180 g/W, Decelerations & Accelerations (180g/W is equivalent to 300 lb/hp) to determine the approximate start and end points of the climbing lane, along with the following recommendations:

- Where a climbing lane would otherwise be located in an expensive cut or fill, it may be more cost-effective to substitute with a passing lane before or after the grade section.
- The TAC Geometric Design Guide for Canadian Roads (Section 2.1.8) method on long hills may be used to determine if the truck climbing lane is warranted.
- Intersections should be avoided within the climbing lane, particularly on the left side of the climbing lanes and at the following locations on both sides: within the decision sight distance (DSD) coming up to the merge end of the climbing lane, or within 300 metres past the diverge taper. Refer to Table 1.2.5.6 - TAC Geometric Design Guide for Canadian Roads for DSD. Where an intersection within the climbing lane section cannot be avoided, the intersection should be in the middle of the climbing lane section and away from the merge and diverge areas where weaving manoeuvres are occurring and driver workload is high. This is normally accomplished by moving the diverge location 300 metres prior to the intersection or 100 metres past it and/or extending the climbing lane merge beyond the intersection for a distance equivalent to the DSD.
- Where traffic volumes are moderate to high (SADT greater than 1000 veh/day), driver reaction to short climbing lanes is generally negative. The minimum climbing lane should allow about 30 seconds of passing opportunity, which is equivalent to 700 m at 80 km/h. At

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traffic volumes lower than 1000 SADT a minimum climbing length of 500 m is recommended.

- The minimum climbing lane width is 3.6 m. The shoulder adjacent to the climbing lane may be up to 1.0 m less than the shoulder adjacent to the 2-lane section, but no less than 1.5 m. If this is part of a staged development to 4-lane, the climbing lane shoulder width should match the ultimate 4-lane shoulder width.
- The diverge taper, merge taper and signing shall be done in accordance with the Ministry's Manual of Standard Traffic Signs & Pavement Markings<sup>2</sup>. Advance signing of a climbing lane ahead should be considered to encourage drivers to wait, rather than perform a hazardous passing manoeuvre.
- DSD ahead to the middle of the diverge taper is desirable from an operation perspective, but not critical from a safety perspective. Good sight distance means that the climbing lane will be used more effectively since traffic can see the climbing lane coming, encouraging earlier separation of slow and fast moving vehicles into their respective lanes.
- Sight distance from the start of the merge taper ahead, should be equal to the minimum barrier line passing sight distance in the Manual of Standard Traffic Signs & Pavement Markings. This allows for a pass initiated at the end of the climbing lane to be safely completed or aborted if the overtaking vehicle is forced into the opposing lane.

## 920.04 MULTILANE HIGHWAYS

Climbing lanes on multilane highways also serve to separate slower vehicles from faster ones and thereby help maintain a high level of service on long grades. The analysis and determination if multilane climbing lanes are warranted is a Planning function that follows the methodology outline in page 20-28 of the Highway Capacity Manual, TRB (HCM 2000)<sup>3</sup>. A drop of one level of service or a speed reduction of 15 km/h on the upgrade is an indicator that a climbing lane may be required. This should be verified with an operational analysis of the approach segment and the upgrade. Should the analysis indicate that an additional lane is required on the upgrade, a climbing lane is warranted. The location and design of climbing lane on multilane highways follows the same Guidelines as for two lane. The corridor strategy is determined by the Ministry's regional planning staff. Detailed design rests with the Regional Design staff or design consultant. Close co-operation is required between planning and design as a team to ensure that the planning objectives are maintained as the design options are evaluated and selected. This close co-operation will improve the likelihood of funding approval.

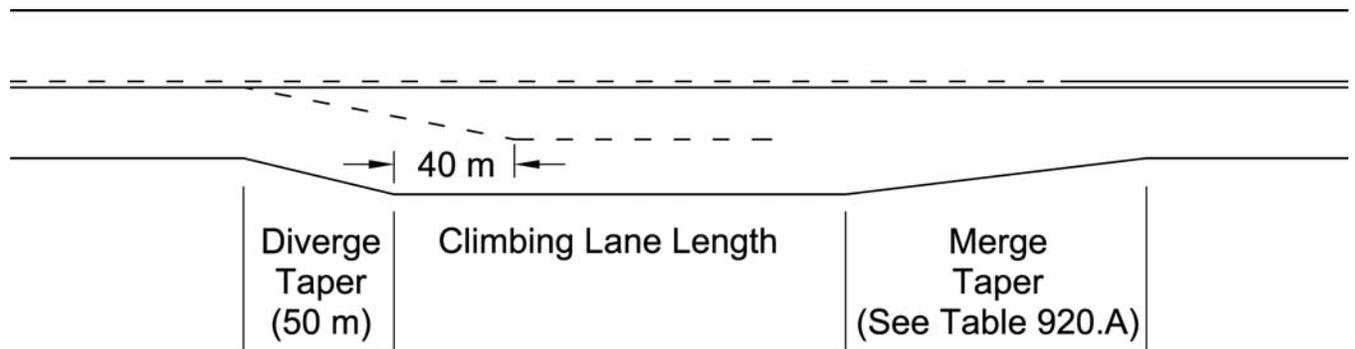
<sup>1</sup> Lyall, Peter D. and Jagannathan, R "Auxiliary Lane Warrants for Two-Lane Highways, Volume I: Report", ADI Limited, Victoria, BC. 1993..

<sup>2</sup> "Manual of Standard Traffic Signs & Pavement Markings", BC Ministry of Transportation, Engineering Branch.

<sup>3</sup> Highway Capacity Manual, Transportation Research Board (HCM2000 - Metric).

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**Figure 920.A Typical Climbing Lane Configuration**



See the Ministry’s Manual of Standard Traffic Signs & Pavement Markings<sup>2</sup> for additional information on Sign Placement and opposing lane passing restriction criteria.

**Table 920.A Merge Taper Lengths**

Posted Speed Limit (km/h)	Merge Taper (m)
50	110
60	130
70	150
80	175
90	195
100	215
110	240

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## 930

# PASSING LANE WARRANTS AND DESIGN

### 930.01 INTRODUCTION

Passing lanes are auxiliary lanes designed to improve passing opportunity on two lane highways except where an auxiliary lane is warranted by grades alone, in which cases climbing lanes are used (refer to Section 920.02).

Regional Planning performs corridor reviews for the purpose of maintaining or upgrading the quality of the provincial road network. Passing lanes are used to upgrade the level of service on a two lane highway where four laning is not contemplated at least at this stage of the planning. When the technical warrants have been met the general procedure should be to: 1) determine the optimum planning and design parameters; 2) estimate the costs of providing the passing lane; and 3) do a benefit-cost analysis. At the detailed design stage, design options are further analyzed for cost and operational efficiency. During this fine tuning exercise, the co-operation between regional planning staff and the designer is crucial to strike a balance between the planning objectives and the design/construction/operation realities.

Passing opportunity on a two lane highway is mainly governed by sight distance and traffic in the opposing direction. When there is insufficient passing opportunity, queues or platoons begin to build up, increasing driver frustration and workload which leads to an increase in risk taking manoeuvres and serious, high speed accidents. Conditions which lead to this type of platoon buildup requiring the consideration of auxiliary passing lanes include:

- long stretches with no-passing opportunities;
- circuitous alignment in rolling or mountainous terrain;
- sparsely developed local street network thereby forcing slow moving traffic to use the highway;
- a high percentage of long distance, high speed trips mixed with slow moving vehicles;

- a significant percentage of slow moving vehicles (heavy trucks & RVs) generating platoons; and
- traffic volumes high enough to restrict passing but too low to warrant widening to four lanes.

### 930.02 LENGTH OF PASSING LANES

Analysis conducted by Harwood <sup>1</sup> on existing U.S. passing lanes shows that the most cost effective length for passing lanes increases with flow rate as follows:

**Table 930.A Passing Lane Lengths**

One-way Flow Rate (vph)	Optimal Passing Lane Length (km)
100	0.8
200	0.8 - 1.2
400	1.2 - 1.6
700	1.6 - 3.2

Some jurisdictions use a consistent 2.0 km length regardless of traffic volume. Although this is desirable, it is often not possible in BC due to roadside development or terrain constraints. It is recommended that 2.0 km be used where possible but shorter passing lanes be considered where necessary. The lane should allow for at least 30 seconds of passing opportunity in order to disperse platoons of 4 to 6 vehicles.

### 930.03 LANE FREQUENCY

Passing lane frequency (LF) 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. Passing lane spacing is the distance from the end of one auxiliary lane to the start of the next in the same direction.

Establishing the need for passing lane frequency is helpful prior to determining potential locations. It

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is also an indication of how practical it is to achieve desired levels of service. If passing lanes are required at very short intervals to maintain a desired level of service, it is an indication that alternatives to passing lanes should be considered.

The desired lane frequency varies depending on:

- passing lane length;
- traffic volumes;
- traffic composition; and
- downstream passing opportunities.

Following are some typical passing lane spacings (end of one lane to the start of the next) given as a function of AADT:

**Table 930.B PASSING LANE SPACINGS**

AADT	Spacing between passing lanes (km)
1001 -3000	9.6
3001 - 5000	8.0
5001 - 7000	6.4
7001 - 9000	4.4
>9000	4.0

**Notes:**

1. Minimum spacing between auxiliary passing lanes is a function of the time it takes for platoons to re-form. This is the basis of Table 930.B. The individual passing lane length includes tapers.
2. Low volume roads which may not warrant passing lanes based on the above criteria may still require some passing opportunities in the form of passing lanes or slow moving vehicle pullouts (see Section 910) if the highway has extended no passing zones. As a guideline, vehicles should have either a passing zone or a passing lane or slow moving vehicle pullout every 10 minutes to prevent drivers from overtaking in a no passing zone. Passing Lanes are auxiliary facilities; passing zones are locations where sight distance permits overtaking by use of the opposing direction lane and are marked with dashed lines.

**930.04 LOCATION GUIDELINES**

Locate individual passing lanes to ensure maximum safety and operational benefits from the investment. The designer should strive to follow these guidelines:

- Locate passing lanes where the minimum feasible construction cost occurs (avoid large cuts and fills, particularly in rock), subject to other constraints.
- Intersections should be avoided within the passing lane, particularly on the left side and in the vicinity of the merge and diverge tapers. Avoid intersections within the decision sight distance (DSD) upstream of the merge end of the passing lane, or within 300 metres downstream of the diverge taper. Refer to Table 1.2.5.6 - TAC Geometric Design Guide for Canadian Roads for DSD.
- When an intersection in the passing lane section cannot be avoided, the intersection should be in the middle of the passing lane section away from the merge and diverge areas where other weaving manoeuvres are occurring and driver workload is high. The intersection should have a separate left turn lane regardless of traffic volume since a stopped left turn vehicle in the passing lane represents a high hazard to overtaking traffic. "T" Intersections on the passing lane side are more desirable than intersections on the opposing side; they do not generate left turn movements to or from the fast lane.
- Minimum DSD ahead to the middle of the diverge taper is desirable from an operation perspective, but not critical from a safety perspective. Good sight distance means the passing lane will be used more effectively since traffic can see the passing lane coming, encouraging earlier separation of slow and fast moving vehicles into their respective lanes.
- The sight distance to the middle of the merge taper should be at least equal to the minimum decision sight distance to allow for an overtaking vehicle to either complete a pass or adjust speed to that of the slower vehicle. The

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termination point should be visible to approaching traffic and allow a smooth and safe merge between slow and fast vehicle streams.

- Sight distance from the start of the merge taper ahead, should be equal to the minimum barrier line passing sight distance in the Ministry's Manual of Standard Traffic Signs & Pavement Markings<sup>2</sup>. This allows for a pass initiated at the end of the climbing lane to be safely completed if the overtaking vehicle is forced into the opposing lane.

Note that Barrier Line Sight distance is not the same as design passing sight distance. The former includes distance traveled by the overtaking vehicle while encroaching on the opposing lane plus half the distance traveled by an opposing vehicle while design passing sight distance includes barrier line sight distance plus a component for the initial decision/acceleration phase.

- Where possible, develop the diverge taper around a long flat horizontal curve. This facilitates separation of the fast and slow streams of traffic and does not take away from any existing passing opportunity. Left hand curves offer the overtaking drivers a better view into the passing lane around the impeding vehicle. Right hand curves do not have the same sight distance, but do lead slower vehicles naturally into the slow lane since the normal driving tendency is to steer to the inside of the curve.
- Passing lanes after a long no-passing zone are more effective than one constructed before it. The upstream no passing zone causes platoon buildup prior to the passing lane and downstream passing opportunities help platoons to remain dispersed longer.
- The addition of passing lanes should not be detrimental to the passing opportunities for the opposing direction. Avoid passing lanes in locations where passing is already permitted by markings, unless the passing opportunity is significantly lessened due to high opposing volumes. The location of the passing lane should appear logical to the driver; its value is

more obvious to the driver at locations where normal passing sight distance is restricted.

- Passing lanes are less effective where passing opportunity is already high. Horizontal curves and upgrades where no passing prevails are good locations.
- Passing lanes provide little benefit when constructed on long tangent sections and on long downgrades with low traffic volumes (AADT<3000) and low percentages of heavy trucks. In these cases, platoon leaders on such sections tend to speed up or not pull over, limiting the benefits from the passing lane.
- Passing lanes should be placed leading away from rather than into areas of traffic congestion. When placed on the outbound direction from a town (or development) they are helpful in dispersing platoons which have built up within the town. Passing lanes on the inbound direction just before a town are less effective and may also be undesirable by encouraging high speed passing just before a reduced speed zone.
- Avoid passing lanes near four-lane highway sections which effectively serve the same purpose.
- Passing lanes in the uphill direction of a highway on a sustained grade section are more effective than one on a level grade because of the greater speed differentials.
- Reduced speed (sub standard) curves, should be avoided in passing lane sections since there is a tendency for traffic to speed up in these sections. Horizontal curves should be at least equal to the minimum radius for the design speed of the highway (see Section 330).
- Physical constraints such as bridges and culverts should be avoided due to the additional cost and the lack of a continuous shoulder through the passing lane section.
- The total length of all auxiliary lanes in one direction should be less than half the roadway section length. Also, the passing opportunity should be equal in both directions.

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### 930.05 GEOMETRY

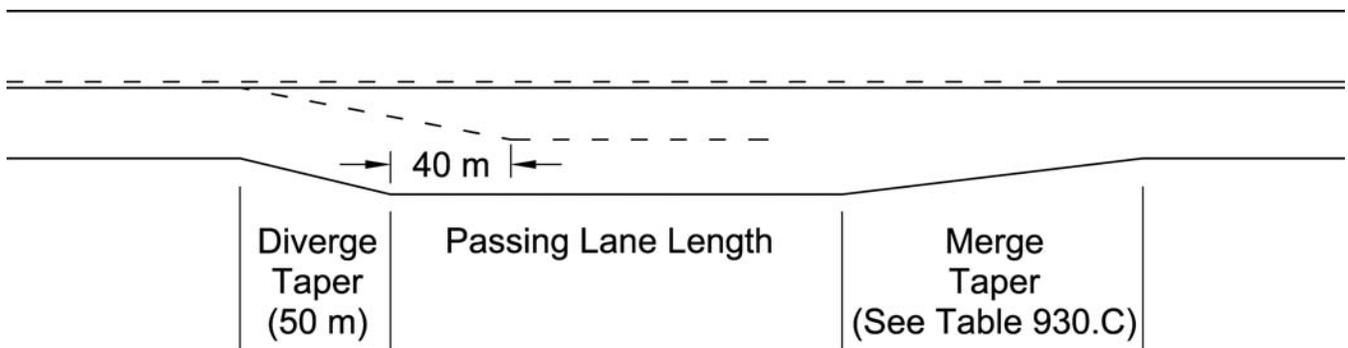
Geometric design standards should be consistent with the following MoTI guidelines and practice:

- The desirable length of passing lanes is between 1.5 km and 2.0 km. This range is long enough to be adequate for dispersing queues while still being short enough to be cost effective.
- The minimum lane width is 3.6 m. The shoulder adjacent to the passing lane should desirably be the same as the shoulder adjacent to the 2-lane section. If the shoulder must be reduced, the reduction should not exceed 1.0 m and the remaining width should be no less than 1.5 m.

If this is part of a staged development to 4-lane, the passing lane shoulder width should match the ultimate 4-lane shoulder width.

- The diverge taper, merge taper and signing shall be done in accordance with the Ministry’s Manual of Standard Traffic Signs & Pavement Markings<sup>2</sup>. Advance signing (I-082-1 and I-082-2 in Manual of Standard Traffic Signs & Pavement Markings) 2 km ahead of a passing lane ahead should be used to advise drivers to wait, rather than perform a hazardous pass. Benefit-cost analysis assumes a 3% reduction in accidents for this 2 km, due to the advanced signing alone.

**Figure 930.A Typical Passing Lane Configuration**



**Table 930.C Merge Taper Lengths**

Posted Speed Limit (km/h)	Merge Taper (m)
50	110
60	130
70	150
80	175
90	195
100	215
110	240

See the Ministry’s Manual of Standard Traffic Signs & Pavement Markings<sup>2</sup> for additional information on Sign Placement and opposing lane passing restriction criteria.

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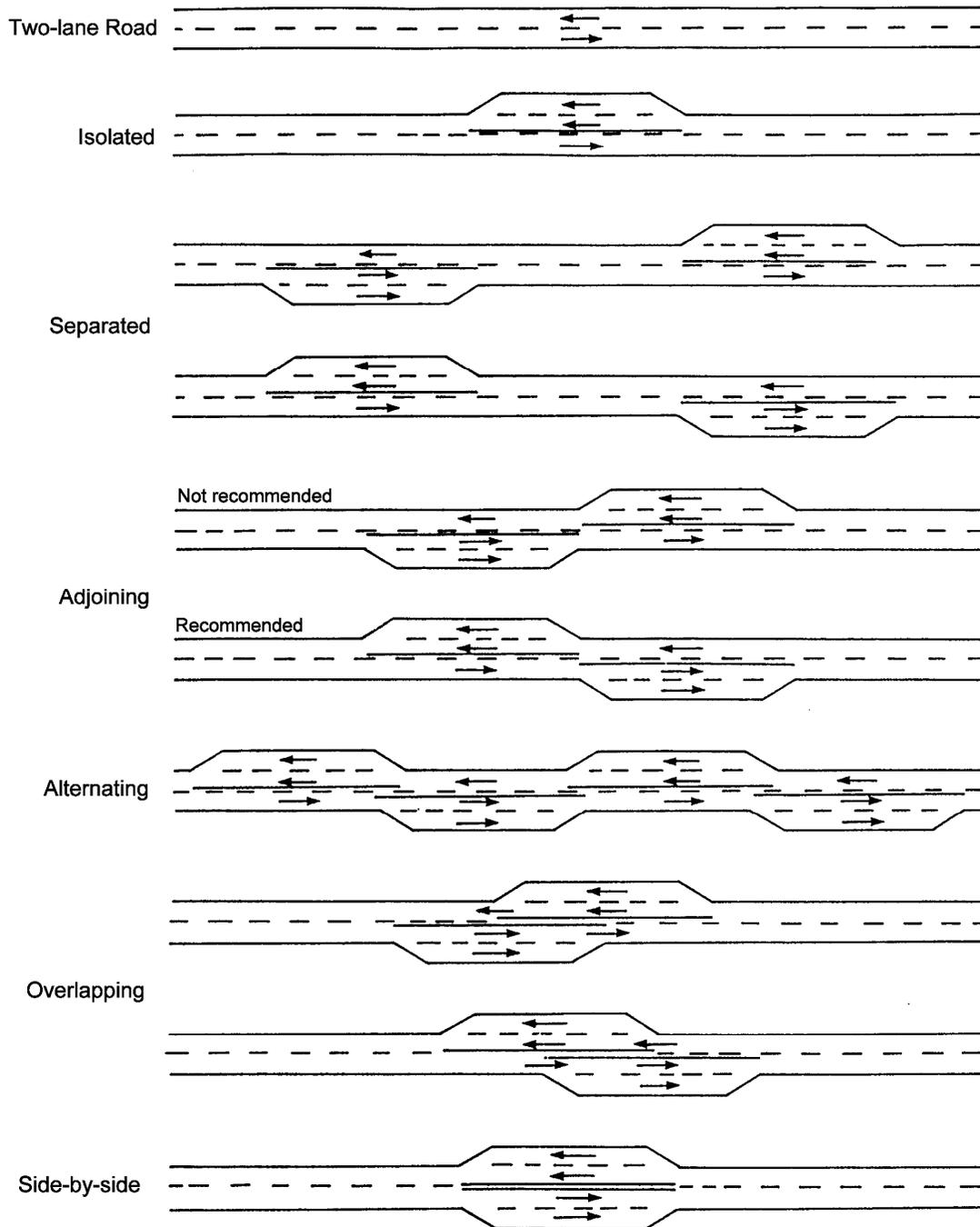
## 930.06 GUIDELINES FOR A SYSTEM OF PASSING LANES

On highways that are not constrained by development or terrain, there may be several sections that satisfy some or all of the location guidelines. The selection of an overall workable combination of passing lanes for both directions of traffic from these sections is an iterative process. The design options should be tested for their overall operational effectiveness as a system. **Figure 930.B** shows schematically some ways to combine passing lanes in both directions as a unified system. Following is a suggested method to develop optional arrangements for review as a system:

- Initially, identify all potential passing lane locations in both directions, irrespective of desired lane frequency.
- With the climbing lane locations fixed and potential passing lane locations identified, select combinations of auxiliary lanes at (or close to) the desired passing lane frequency taking into account any existing auxiliary lanes. The frequency should be no less than four kilometres (including the length of auxiliary lane) apart.
- It is generally desirable to stagger opposing direction passing lanes to avoid the mistaken impression of a 4 lane highway. Some overlap is acceptable. Short four-lane sections are appropriate in valley sections where there is no other option or where the whole section would form part of an ultimate four-laning scheme.
- Do not overlap opposing auxiliary lanes through major intersections. This would require additional turning lanes resulting in a five-lane cross section (two-lane highway, two auxiliary lanes, and one turning lane) in an otherwise two-lane highway template. These types of intersections are confusing to through traffic and are also very difficult for left turn minor road traffic to negotiate due to number of conflicting lane movements on the major road.
- Where possible, place opposing auxiliary lanes tail-to-tail rather than head-to-head (the tail is the diverge). In the tail-to-tail configuration, the opposing direction auxiliary lane restricts advancing passing maneuvers upstream of the advancing lane rather than downstream. The head-to-head configuration may also be a safety problem in winter time when pavement markings are hidden by snow resulting in vehicles traveling in the oncoming passing lane.
- When dealing with high traffic volumes and limited passing opportunities, avoid placing a single auxiliary lane in a long section favoring one direction of traffic at the expense of the opposing traffic. This may cause a race-track effect, where aggressive drivers tend to speed up to make use of the only passing opportunity available. Staged development of an auxiliary lane system may result in this situation, but it should be avoided in the ultimate development of the auxiliary lane system.
- Try to achieve balanced overall passing opportunities for both directions.

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**Figure 930.B Alternative Configurations for Passing Lanes:**



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## 930.07 ANALYTICAL METHOD FOR A SYSTEM OF PASSING LANES

The passing lane analysis, which follows is performed after climbing lane warrants have been considered. This is an estimation of level of service for the existing highway at design year using regression equations. These equations predict the percent following as a function of Assured Passing Opportunity (APO). The regression equations were derived from TRARR simulations.

The APO factor is defined as the percentage of time when one vehicle can safely pass another without restriction either by inadequate sight distance or the presence of opposing traffic. It is often used as the level of service measure for passing lane studies since it relates directly to the role of passing zones in reducing vehicle platoons. APO is calculated separately for each direction as:

$$APO = (PZL/L) * HF$$

where

PZL	= Passing Zone Length (km)
HF	= Headway Factor (%)
L	= Length of the highway segment (km)

PZL for a direction is the length of highway within a highway segment L which has passing zones (broken lines) and can be determined through viewing the Ministry of Transportation's photolog imaging system on the Internet. To obtain access to the photolog Internet site, contact the Data Program Coordinator, Highway Planning Branch, Victoria. For highways which have not yet been constructed, PZL must be estimated from plans and profiles using the Manual of Standard Traffic Signs & Pavement Markings<sup>2</sup> standards for barrier line passing sight distance. This dimension is different from Design Passing Sight Distance and is defined in the Manual of Standard Traffic Signs & Pavement Markings.

Headway factor (HF) is the percentage of time when headway between successive vehicles in the opposing lane is greater than 25 seconds. The 25 seconds criterion is based on the time taken for an overtaking vehicle for the initial maneuver (5 sec.) plus encroachment on the opposing lane (10 sec.) plus 5 seconds for an opposing vehicle appearing half way through the overtaking vehicle encroachment phase plus a 5 second clearance to the opposing vehicle at the completion of the pass.

Headway factor is approximated as:

$$HF = \exp(-k * V_{OPP})$$

where  $V_{OPP}$  is the volume of opposing direction traffic (veh/h) and k is a constant dependent on the terrain. For a given opposing volume, the percentage of time with gaps in the opposing direction greater than 25 seconds is typically higher for mountainous terrain than for a level terrain. In mountainous terrain, a higher proportion of vehicles will be in platoons; therefore, long gaps are introduced between platoons.

The following k constants are recommended:

- 0.002 for mountainous terrain (observations on Highway 99 by ADI Limited<sup>5</sup>, and Alberta data),
- 0.004 for rolling terrain (Ontario),
- 0.006 for level terrain or highways with high access volumes (observations in Victoria, BC by ADI Limited).

Planners may wish to use the actual headway factors, measured in the field, for a given highway rather than an estimate based on the k constant. Where there is a high percentage of PZL, the calculated percentage following becomes more sensitive to the choice of k constants. Varying the k constant by 0.001 typically changes the percent following by about 5% (15% is equal to one level of service).

HF may be estimated from some of the Province's Weigh-in-Motion stations or manually by measuring the traffic opposing traffic volume (veh/h) and gaps greater than 25 seconds for groups of about 100 vehicles. The headway factor at the observed flow rate is calculated as follows:

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$$HF = \frac{(\sum(\text{gaps} > 25 \text{ sec}) - 25N)}{T}$$

The main objective is to relate the APO and the total auxiliary passing lane length (ALL) to the percentage of vehicles following (%FOLL) and thus, the level of service (LOS).

The frequency and length of platoons on a two-lane highway depends on the amount of available APO and the advancing traffic volume ( $V_{adv}$ ). Regression analysis of percent following results from TRARR resulted in the following equations for the three terrain types:

<b>%FOLL =</b>	<b>0.000365 x <math>V_{adv}</math> - 0.89278 x %APO +0.53</b>	<b>For Level terrain; <math>R^2 = 92\%</math></b>
<b>%FOLL =</b>	<b>0.000346 x <math>V_{adv}</math> - 1.09273 x %APO +0.58</b>	<b>For Rolling terrain; <math>R^2 = 94\%</math></b>
<b>%FOLL =</b>	<b>0.000330 x <math>V_{adv}</math> - 1.86374 x %APO +0.67</b>	<b>For Mountainous terrain; <math>R^2 = 92\%</math></b>

where %APO and %FOLL are expressed as decimals. The equations are shown as the upper graphs of Figures 930.C to 930.F. Note that these equations assume there are no existing auxiliary lanes.

Simulations were then repeated on the same road sections with the addition of auxiliary lanes and these results are also shown in the lower graphs of Figures 930.C to 930.F. The total length of auxiliary lanes (ALL) in a direction is expressed in these graphs as a percentage of section length (L). These graphs can be used to estimate the effect of existing auxiliary passing lanes in reducing percent following.

### 930.08 Estimation of Level of Service

The method to estimate level of service for each direction is as follows:

1. Calculate the APO for the design year volume. The passing zone length PZL in this equation is the length of passing lines in the advancing direction and the headway factor HF is calculated as shown above.

2. Use the calculated APO and the advancing traffic volume to estimate percent following from the upper graphs of **Figures 930.C to 930.F** as appropriate. This is the percent following in the absence of any auxiliary lanes.
3. If there are existing auxiliary lanes, calculate %ALL = ALL / L where ALL is the length of auxiliary lanes.
4. Read the reduction in percentage following using %ALL and the adjustment factors in the lower graphs of **Figures 930.C to 930.F**.
5. Apply the appropriate reduction to the original estimated percent following to get the new value which takes into account the existing auxiliary lanes.
6. Obtain the level of service corresponding to estimated percent following using **Table 930.D**.

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## 930.09 Warrants

The Highway Capacity Manual (HCM3) defines “percentage following” (%FOLL) as the percentage of vehicles which are traveling in platoons at headways of less than 5 seconds. The level of service (LOS) of 2-lane highways can be related to the %FOLL, as shown in the following table:

**Table 930.D Percentage Following and Level of Service**

%FOLL	LOS	Traffic characteristics
$0 \leq \%FOLL < 30$	A	Highest quality of traffic service Drivers at their desired speeds Passing demand well below passing capacity
$30 \leq \%FOLL \leq 45$	B	Significant passing demand Passing demand approximately equals passing capacity No noticeable increase in platoon sizes
$45 < \%FOLL \leq 60$	C	Noticeable increase in platoon formation and platoon size Increased frequency of passing impediment Passing demand exceeds passing capacity
$60 < \%FOLL \leq 75$	D	Passing demand increases dramatically Passing capacity approaches zero Mean platoon sizes of 5-10 Fraction of passing zones has little influence on passing
$75 < \%FOLL < 100$	E	Passing is virtually impossible Platooning becomes intense Highest attainable volume defines the capacity of the highway
$\%FOLL = 100$	F	Heavily congested flow Traffic demand exceeds capacity Speeds well below capacity speed

Percent Following		Inference
Rural Arterial Highways Design goal = LOS C (i.e. <60% following)	Rural Collector Roads Design goal = LOS D (i.e. <75% following)	
< 45%	< 60%	Passing lanes are of low priority and no further consideration is required.
45% to 60%	60% to 75%	Need for passing lanes is marginal. Accident history review may justify improvements.
> 60%	> 75%	Passing lanes are warranted. Reg'l Design & Planning Staff to determine optimal/possible locations and cost estimates.

Two Examples follow on the next page.

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**Example 1**

Given:

Length of study area, L = 40 km  
 Mountainous terrain, no auxiliary lanes, 1.4 km of passing zones  
 DHV = 562 vph, 85:15 split,  $V_{adv} = 478$  vph,  
 $V_{opp} = 84$  vph  
 Design goal of LOS C/D interface (60% Foll)

Calculate:

Headway Factor

$$HF = e^{-0.002V_{OPP}} = e^{-0.002 \times 84} = 0.845$$

PZL = 1.4 km

$$(APO) = (PZL/L) \times HF = (1.4/40) \times 0.845 = 0.030$$

From the mountainous terrain equation:

$$\begin{aligned} \%FOLL &= 0.000330(V_{adv}) - 1.86374(APO) + 0.67 \\ &= 0.000330(478) - 1.86374(0.030) + 0.67 \\ &= 0.77 \text{ or } 77\% \end{aligned}$$

Since %FOLL at the design goal is 60%, auxiliary lanes are required to reduce percent following from 0.77 to 0.60; i.e. by  $[(0.77-0.60)/0.77] = 22\%$ .

From Figure 930.F, a 22% reduction in percent following at a  $V_{adv}$  of 478 vph requires about 28% ALL which corresponds to  $0.28 \times 40 = 11.2$  km of passing lanes.

Assuming a typical passing lane length of 2.0 km the desired lane frequency is:

$$LF = L/(ALL/2.0) = 40/(11.2/2) = 7.1 \text{ km}$$

*The upper graph of Figure 930.C shows the use of Figure 930.F for this example.*

**Example 2**

Given:

Length of study area, L = 40 km  
 Mountainous terrain, 7.7 km of auxiliary lanes, 1.4 km of passing zones  
 DHV = 758 vph, 85:15 split,  $V_{adv} = 644$  vph,  
 $V_{opp} = 114$  vph  
 Design goal of LOS C/D interface (60% Foll)

Calculate:

Headway Factor

$$HF = e^{-0.002V_{OPP}} = e^{-0.002 \times 114} = 0.796$$

PZL = 1.4 km

$$(APO) = (PZL/L) \times HF = (1.4/40) \times 0.796 = 0.028$$

From the mountainous terrain equation:

$$\begin{aligned} \%FOLL &= 0.000330(V_{adv}) - 1.86374(APO) + 0.67 \\ &= 0.000330(644) - 1.86374(0.028) + 0.67 \\ &= 0.83 \text{ or } 83\% \end{aligned}$$

Existing Percentage Auxiliary Lane Length (%ALL) =  $7.7/40 = 19\%$

From the lower graph of Figure 930.F, we get:

25%ALL → 17% reduction in percent following  
 0%ALL → 0% reduction in percent following

Therefore, from interpolation:

$$19\%ALL \rightarrow 19 \times 17/25 = 13\% \text{ reduction in percent following (Interpolated)}$$

$$\text{Percent following} = 0.83 \times (1 - 0.13) = 72\%$$

Since %FOLL at the design goal is 60%, additional auxiliary lanes are required to reduce percent following from 0.72 to 0.60; i.e. by  $[(0.72-0.60)/0.72] = 17\%$ .

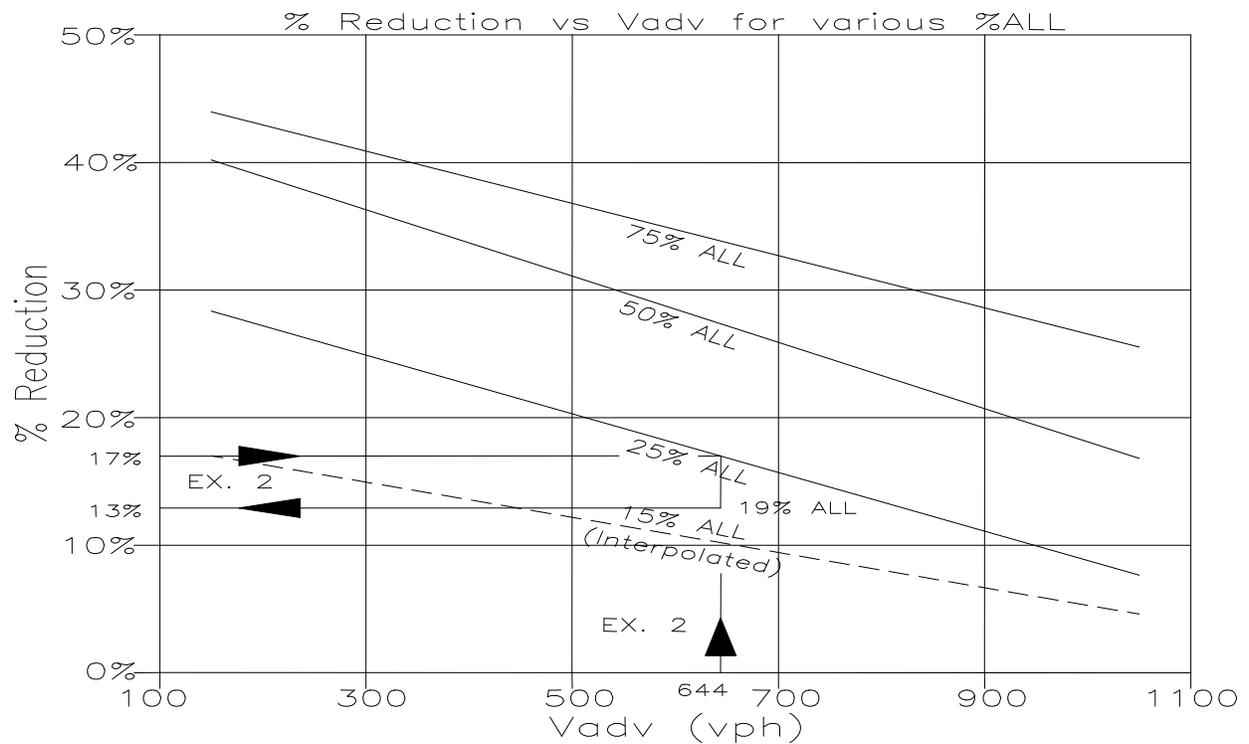
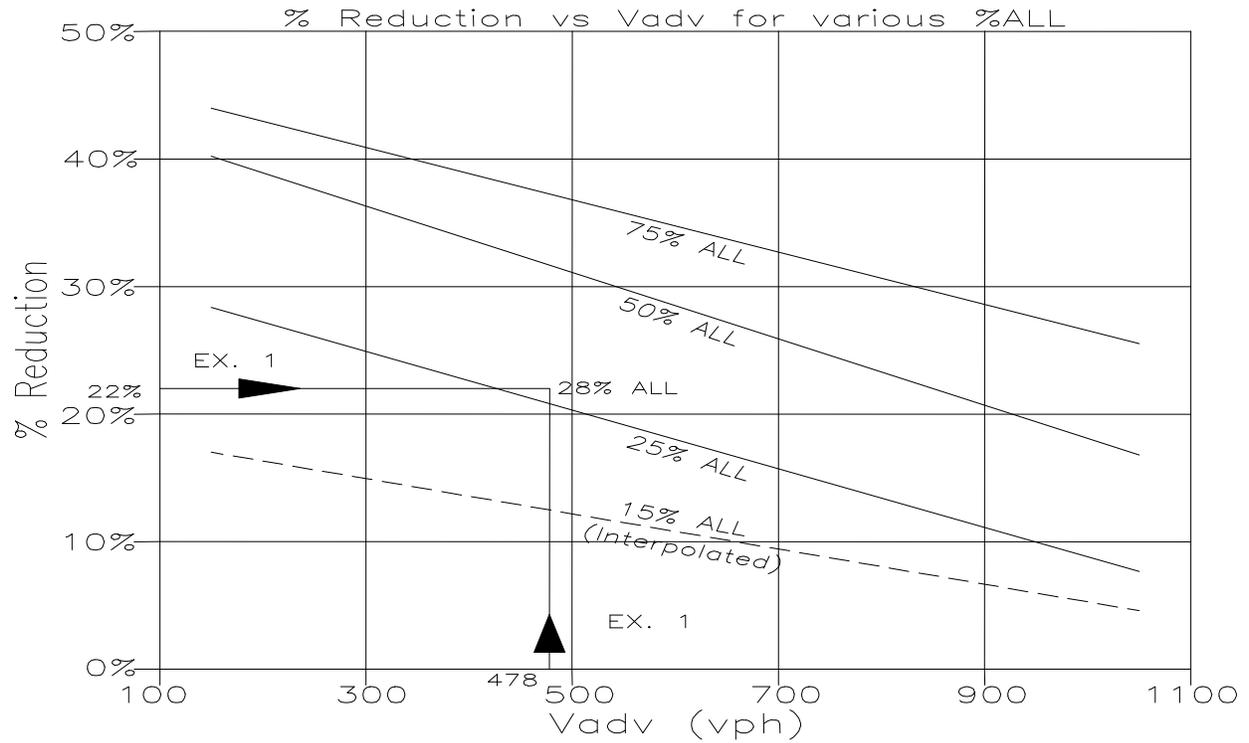
From Figure 930.F, a 17% reduction in percent following at a  $V_{adv}$  of 644 vph requires 25% ALL which corresponds to  $0.25 \times 40 = 10.0$  km of passing lanes, in addition to existing 7.7 km of passing lanes.

Assuming a typical passing lane length of 2.0 km the desired lane frequency is:

$$LF = L/(ALL/2.0) = 40/(17.7/2) = 4.5 \text{ km}$$

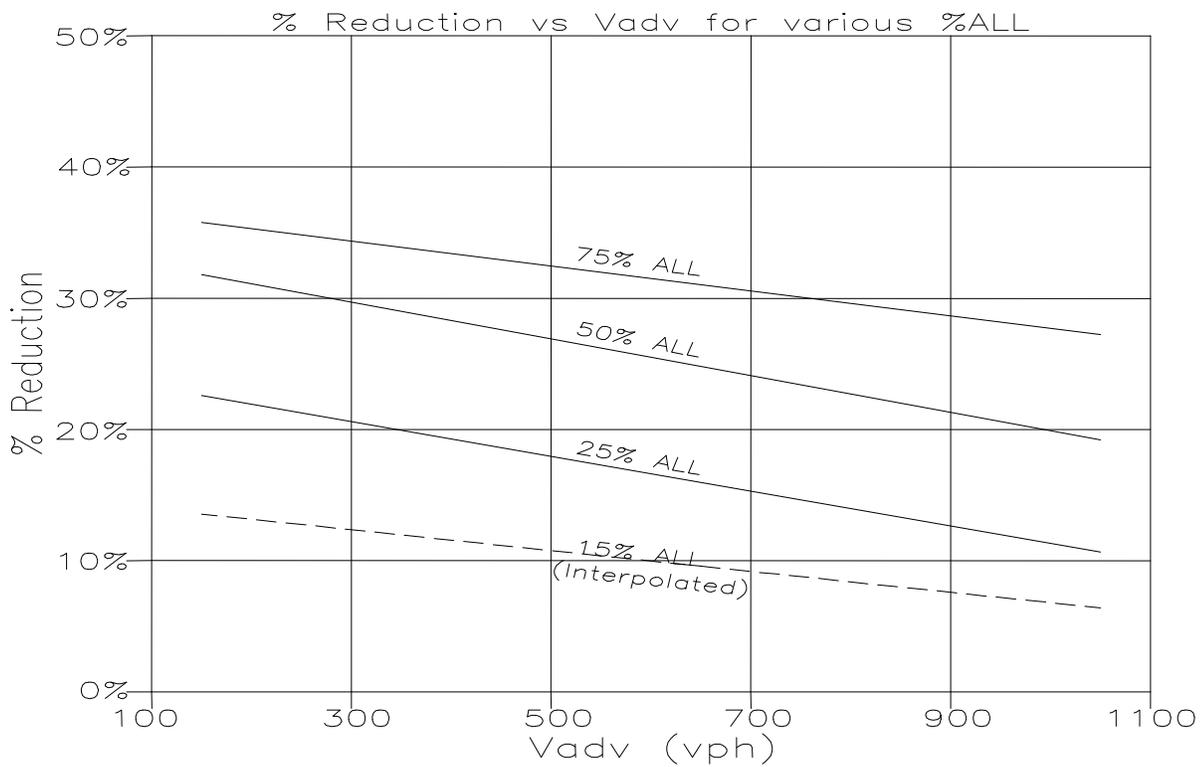
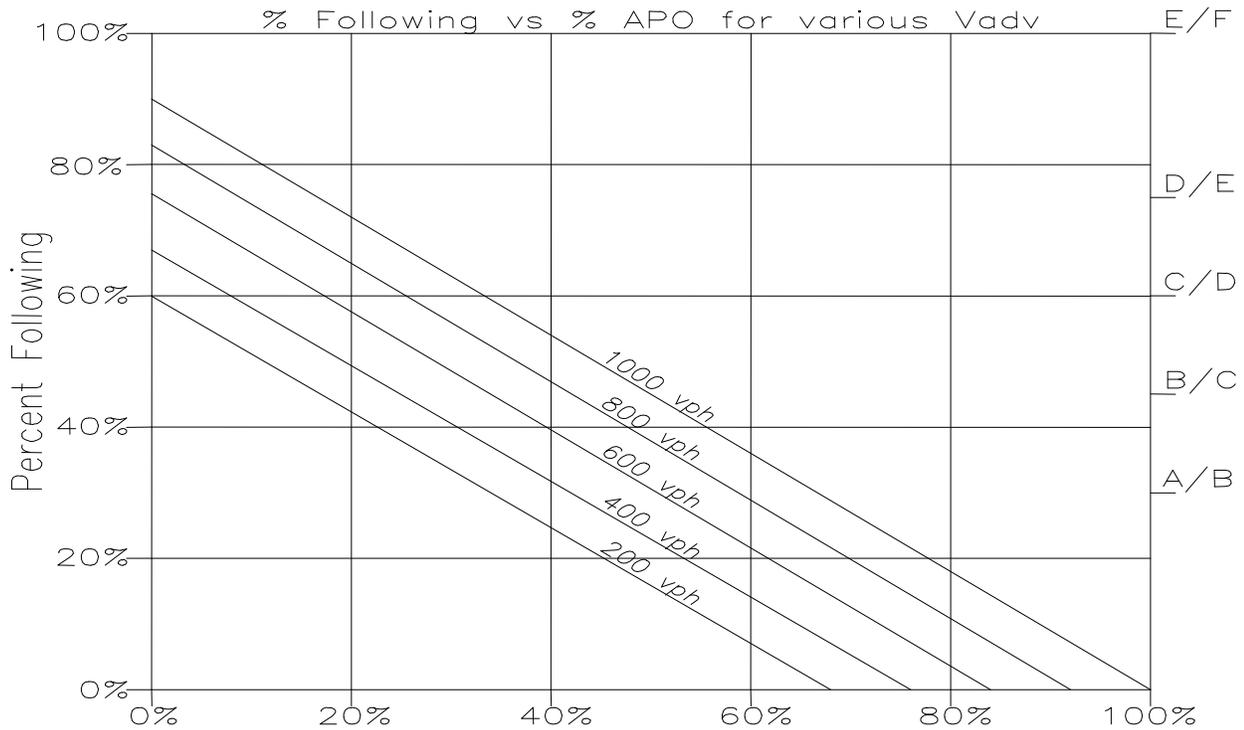
*The lower graph of Figure 930.C shows the use of Figure 930.F for this example.*

Figure 930.C Examples 1 and 2



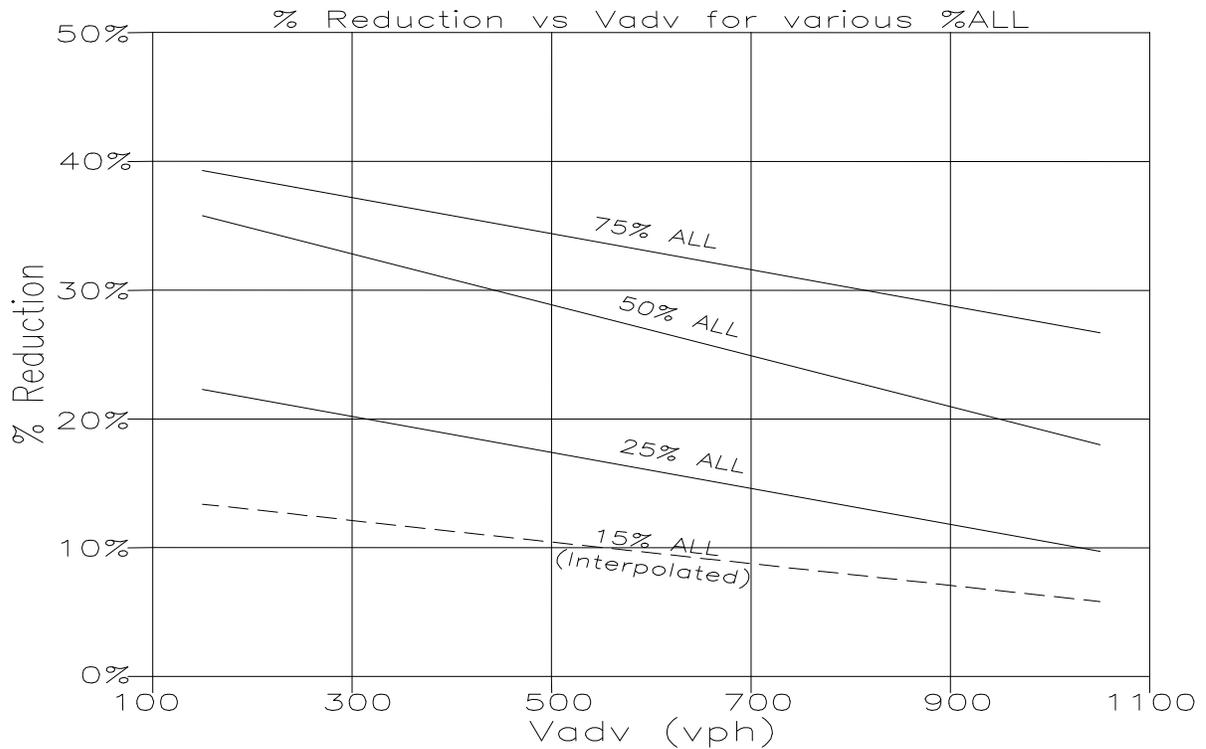
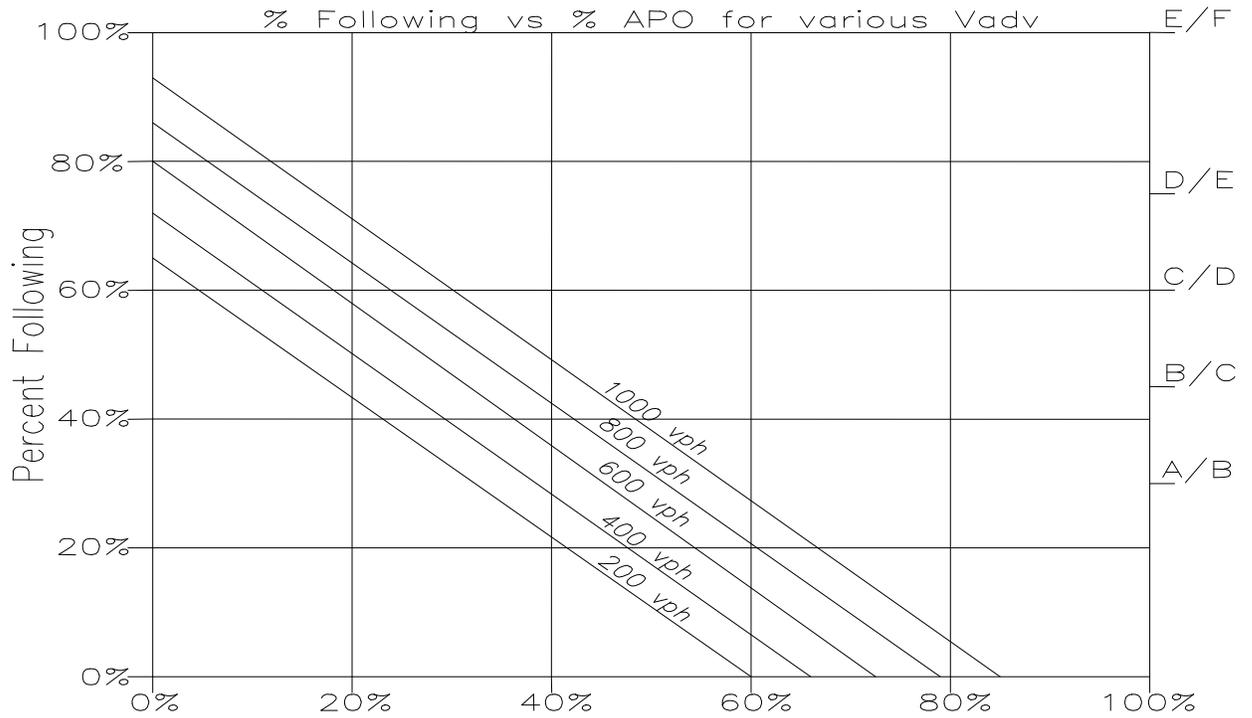
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**Figure 930.D Level Terrain Graphs**



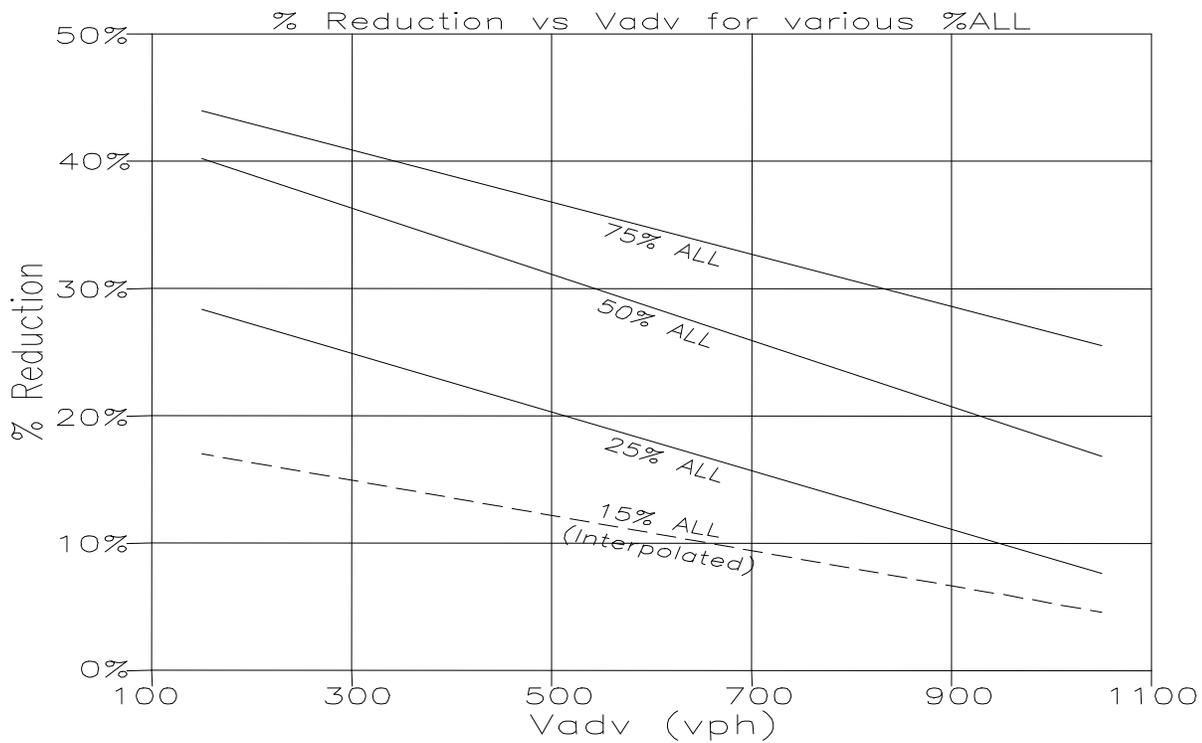
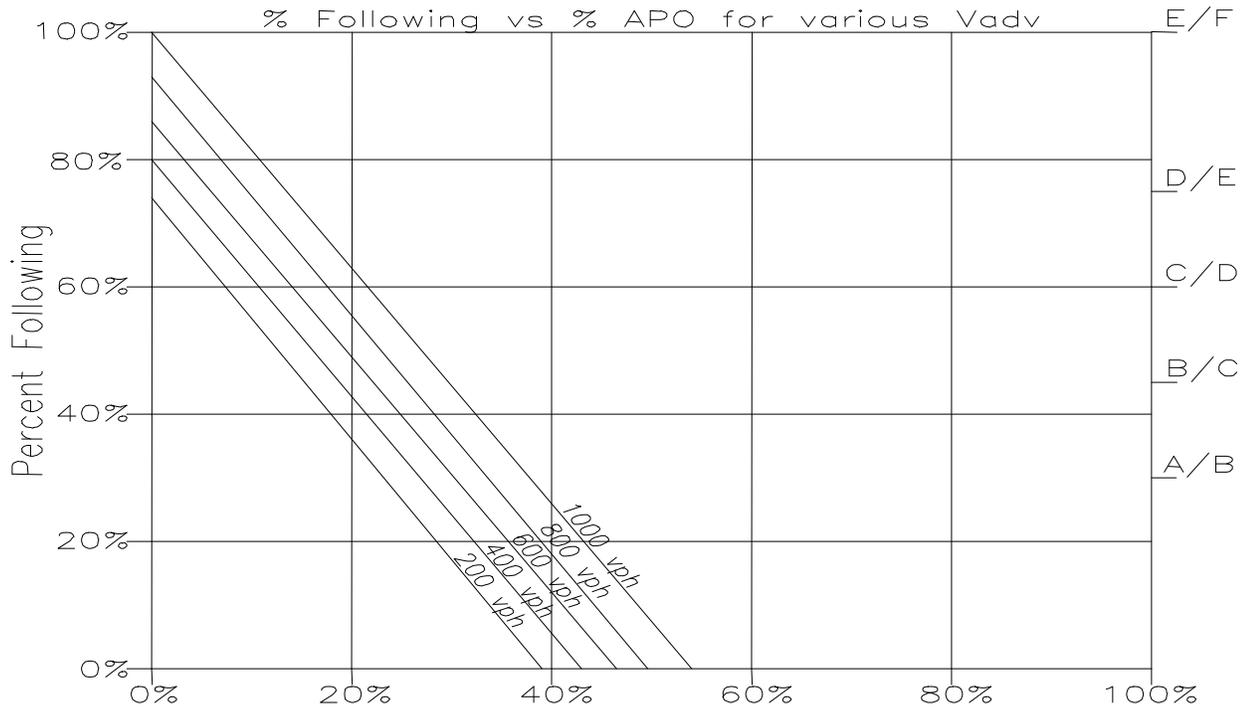
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Figure 930.E Rolling Terrain Graphs



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Figure 930.F Mountainous Terrain Graphs



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## References:

1. Harwood D.W., Hoban C.J., Warren D., "Effective Use of Passing Lanes on Two-Lane Highways", Transportation Research Record 1195, Transportation Research Board, 1988.
2. "Manual of Standard Traffic Signs & Pavement Markings", BC Ministry of Transportation, Engineering Branch.
3. Highway Capacity Manual, Transportation Research Board (HCM2000 - Metric).
4. Lyall, Peter D. and Jagannathan, R "Auxiliary Lane Warrants for Two-Lane Highways, Volume I: Report", ADI Limited, Victoria, BC. 1993.

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## 940 COMMUNITY MAILBOX PULLOUT

### 940.01 INTRODUCTION

This guideline is for locating Community Mailbox Pullouts adjacent to Ministry jurisdiction roadways. This is for highway district approvals staff, regional highway planning and professional services staff, and Ministry staff or consultants working on a highway design project. This is also part of a protocol agreement between Canada Post and the BC Ministry of Transportation and Infrastructure.

The main objective is to ensure that mailbox pullouts do not interfere with the safe and efficient operation of roadways under Ministry jurisdiction.

### 940.02 SITE SELECTION

Some basic rules should be used when selecting a site. These are:

- No Community Mailbox Pullouts are to be installed on divided highways or major arterial highways where access control is exercised (Freeways, Expressways and Controlled Access Highways). The more important the highway, the higher the speed and/or traffic volume; therefore, a Pullout site will have a greater impact on the operation and safety of the roadway.
- Make every effort to install Community Mailbox Pullouts on side roads that access residential subdivisions.
- In urban areas with pedestrian traffic, the preferred location is on a street that has a sidewalk and sufficient road width for on-street parking.
- For all locations, stopping sight distance must be met on the roadway adjacent to the site.
- Give particular care near intersections so as not to interfere with the safe operation of the intersection. Visibility of traffic signs and signals should not be blocked. The site shall not encroach upon auxiliary right and left turn lanes at intersections and the sight triangle.

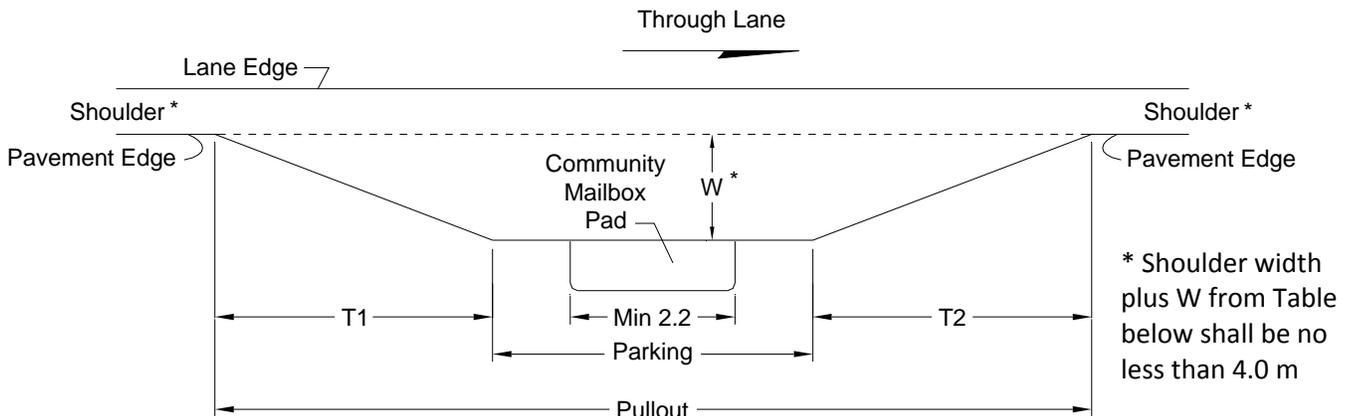
### 940.03 SITE LAYOUT

#### Geometry

- The higher the road classification, the higher the safety requirements are for the Pullout. The site is composed of tapers, parking and a pad area. See **Figure 940.A** and **Table 940.A** for dimensions.
- A 300 mm or larger culvert, as directed by the Ministry Representative, shall be used. The culvert bed shall be cleared, graded and day-lighted as directed by the Ministry Representative.
- Desirably, culvert grade shall be slightly steeper than critical grade (usually 1 to 2.2%). The minimum grade is 0.5% to prevent sedimentation.
- Crossfall for drainage shall be away from the road. Where a site is located on the outside of a curve on a 2-lane road, crossfall may match the crossfall of the road.
- On all roads other than local residential subdivision streets, Community Mailbox Pullout site tapers should not be closer than 30 m to:
  - The start of the taper to a left-turn lane.
  - The start or end of the taper to a right-turn lane or bus bay.
  - The start of the radius to an intersection.
  - The closest road edge of an access or exit not having a radius.
  - The nearest rail of a level railway crossing.
- Community Mailbox Pullout sites shall be designed as separate off-road facilities with their own access and exit driveways, when:
  - The number of mailboxes exceeds 160 along an LVR or RLU, or
  - The number of mailboxes exceeds 100 along an RCU or RAU.
- A minimum 4.4 m shall be provided for vehicle parking. This minimum is relaxed for LVR's and RLU's with speeds of 70 km/h or less. More parking may be required to accommodate mailboxes. See **Table 940.B** for dimensions.

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**Figure 940.A MAILBOX PULLOUT SITE LAYOUT**



**Table 940.A CANADA POST COMMUNITY MAILBOX PULLOUT**

Road Classification	Design Speed (km/h)	Parking (m)	T1 <sup>(3)</sup> (m)	T2 <sup>(3)</sup> (m)	W (m)			
					Under 750	750 to 1500	1501 to 6000	Over 6000
LVR	30-50	2.2 <sup>(1)</sup>	6	6	3.0			
	60-70	2.2 <sup>(1)</sup>	9	9	3.0			
	80	2.2 <sup>(1)</sup>	12	12	3.0			
					Average Annual Daily Traffic (AADT)			
					Under 750	750 to 1500	1501 to 6000	Over 6000
RLU	30-50	2.2 <sup>(1)</sup>	12	12	3.0	3.0	4.0	5.0
	60-70	2.2 <sup>(1)</sup>	18	18	3.0	4.0	5.0	6.0
	80	See Table 940.B	20	20	3.0	4.0	5.0	6.0
RCU	50	See Table 940.B	20	20	3.0	3.0	4.0	5.0
	60		30	30	3.0	3.0	4.0	5.0
	70		40 <sup>(2)</sup>	40 <sup>(2)</sup>	3.0	4.0	5.0	6.0
RAU	80	See Table 940.B	50 <sup>(2)</sup>	50 <sup>(2)</sup>	3.0	4.0	5.0	6.0
	70		60 <sup>(2)</sup>	50	3.0	4.0	5.0	6.0
	80		70 <sup>(2)</sup>	50	3.0	4.0	5.0	6.0

**Notes:**

1. The default minimum parking length is 4.4 m; this is relaxed to 2.2 m for these classes of road.
2. These tapers may be decreased by 10 m for AADT of 751-1500 or by 20 m for AADT less than 750. Notwithstanding the taper adjustment for volumes, the minimum taper for RAU shall be no less than 50 m.
3. If barrier is required, change T1 and T2 taper ratio to suit required Concrete Roadside Barrier flare.

LVR: Low-volume Road (traffic volumes ≤ 200 veh/day)

RLU: Rural Local Undivided

RCU: Rural Collector Undivided

RAU: Rural Arterial Undivided

For all roads with speeds in excess of 70 km/h, the Community Mailbox Pullout site shall be paved. District Development staff have some discretion in this requirement where volumes are less than 750 AADT. If there is no paved shoulder, Districts may ask for a 1.0 m paved shoulder through the site, including the tapers.

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**Table 940.B PARKING REQUIREMENTS**

Number of Modules	Parking Size (minimum)	Comments
1 to 3	4.4 m (2.2 m for certain classes)	See Table 940.A and Note 1 above.
4 to 6	4.4 m	
7 to 9	6.6 m	
9 to 12	8.8 m	

Parking requirements are based on the number of modules. Each module contains a number of individual mailboxes.

### Local Residential Subdivision Streets

- Where the posted speed is 50 km/h or less and where on street parking is permitted, the Community Mailbox pad is located:
  - at least 1.5 metres away from the face of the curb where a curb is in place, or
  - 3.0 metres from the outside edge of the through lane in open ditch sections.
- Where street parking is not permitted, use the LVR Taper and Parking dimensions.
- On the far side of an intersection, the Mailbox Pullout (including taper, where no parking is permitted) must start at least 30 m from the end of the intersection radius. On the near side of an intersection, the Pullout must be at least 10 m from the beginning of the intersection radius. Near driveways, the Pullout must be at least 10 m from both road edges of driveways or as directed by the Ministry Representative.

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## 950 COMMERCIAL VEHICLE INSPECTION SITES

### 950.01 COMMERCIAL VEHICLE INSPECTION STATIONS

*Refer to Technical Circular T-13/06*

The design of commercial vehicle inspection stations shall be in accordance with the current *Commercial Vehicle Inspection Station Design Guide*. All of the roadway features required to access and egress the inspection station site such as ramps, acceleration and deceleration lanes or intersections with the highway shall also be in accordance with the *TAC Geometric Design Guide for Canadian Roads* and the *BC Supplement to TAC Geometric Design Guide*.

The following traffic operation considerations and design criteria should be used when selecting where the commercial vehicle inspection station is to be located:

- design for decision sight distance at the approaches to the intersection or the diverging gores of ramps that provide access to the inspection station;
- design for decision sight distance for the highway section upstream of the location where the traffic exiting the inspection station and the through traffic on the highway complete their merging manoeuvre;
- design exit ramps from the inspection station to allow commercial vehicles to accelerate as close as possible to the posted speed on the highway (desirable minimum shall be posted highway speed less 15 km/h) before the end of the merge point between the traffic exiting the station and the highway traffic;
- in cases where the station is accessed through an intersection, locate the intersection and design the highway alignment at the

approaches to achieve both intersection turning sight distance and decision sight distance for the posted speed;

- design merge, diverge and weave areas for level of service 'C' or better for the horizon year design volume; and
- for a median station installation, perform a safety risk analysis of the highway segment to ensure that it is an appropriate location.

### 950.02 MOBILE WEIGH SCALE PULLOUTS

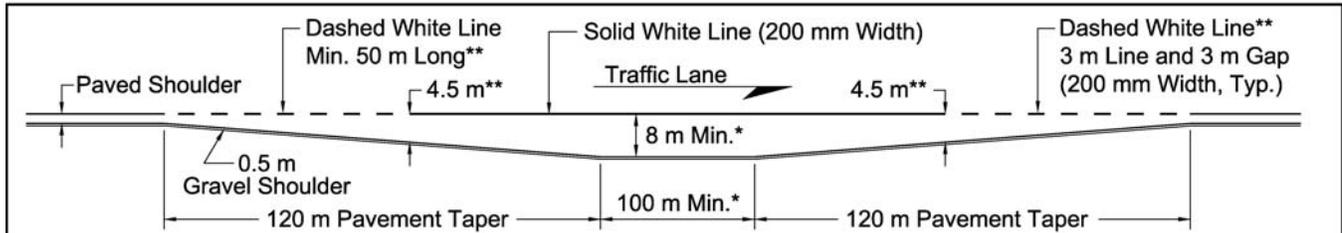
A mobile weigh scale pullout is used for periodic roadside inspections and does not require a building or permanent scale. The site is basically a shoulder widening similar to some highway pull offs for rest stops or brake checks.

Figure 950.A provides guidelines for two layout options that are primarily dependent upon traffic volumes and number of lanes. It is the responsibility of the Regional Traffic Engineer to decide which option is to be used. These guidelines are for all classes of highway except freeways and expressways. The use of mobile weigh scale pullouts is not conducive to maintaining the free flow nature of freeways and expressways. These higher class highways would typically have full sized commercial vehicle inspection stations (refer to 950.01).

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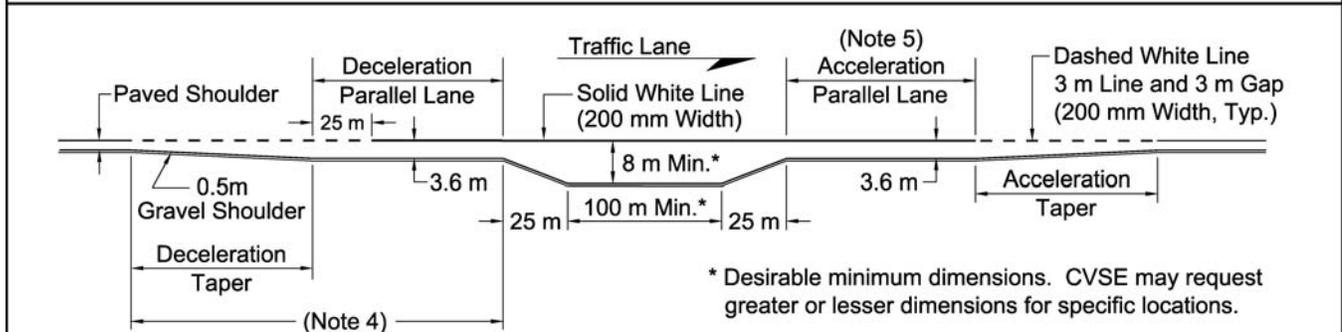
Figure 950.A Mobile Weigh Scales

**GUIDELINES FOR MOBILE WEIGH SCALE PULLOUTS  
(EXCLUDING FREEWAYS AND EXPRESSWAYS)**



FOR TWO LANE HIGHWAYS WITH AADT ≤ 5000 \*\*\*

- \* Desirable minimum dimensions. CVSE may request greater or lesser dimensions for specific locations.
- \*\* Dashed line shall be 50 m long or extend to where the throat is 4.5 m wide, whichever is greater.
- \*\*\* The Regional Traffic Engineer shall decide which layout is to be used. For example, the layout below may be preferred for highways with speeds ≥ 80 km/h to minimize disruption of through traffic. Conversely, the layout above may be used for highways with AADT > 5000 at the discretion of the Regional Traffic Engineer.



\* Desirable minimum dimensions. CVSE may request greater or lesser dimensions for specific locations.

FOR TWO LANE HIGHWAYS WITH AADT > 5000 AND ALL MULTI-LANE HIGHWAYS

DESIGN SPEED (km/h)	DECELERATION PARALLEL LANE P.L. (m)	DECELERATION TAPER (m)	ACCELERATION PARALLEL LANE P.L. (m)	ACCELERATION TAPER (m)
60	25	50	80	55
70	50	50	120	65
80	80	50	165	70
90	120	50	220	80
100	165	50	295	85
110	195	50	375	90

NOTES:

1. For all grades less than 3%, use the P.L. length
2. Deceleration P.L. - for downgrades from 3% to 5%, use P.L. X 1.3
3. Acceleration P.L. - for up grades from 3% to 5%, use P.L. X 1.4
4. The total deceleration lengths (Taper + P.L.) are from 2011 AASHTO Green Book Table 9-22.
5. Acceleration P.L. lengths are based on attaining 85% of the design speed (ref. 2011 AASHTO Green Book Fig. 2-24). Acceleration taper lengths are from TAC Table 2.4.6.5.