GEOMETRIC DESIGN STANDARDS FOR ONTARIO HIGHWAYS

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GEOMETRIC DESIGN STANDARDS FOR ONTARIO HIGHWAYS

INDEX

GLOSSARY

GLOSSARY

Acceleration lane	An auxiliary lane to enable a vehicle entering a roadway to increase speed to merge with through traffic as applied at channelized intersections, or as speed-change lane at interchanges.
Access control	Synonymous with control of access.
Adverse crown	A section with the cross-slope removed to a zero slope. This change in cross-slope is accomplished over the tangent runout.
Assumed speed	The assumed speed for calculating minimum stopping sight distance is based on the 85th percentile wet weather speeds as derived from a ministry study.
Assured passing opportunity	A condition in which a vehicle can safely pass another without restriction either by visibility or opposing traffic.
Average daily traffic (ADT)	The total volume of traffic during a given time period (in whole days) greater than one day and less than one year divided by the number of days in that time period.
Average annual daily traffic (AADT)	The average 24 hour, two-way traffic for the period January 1 st to December 31 st.
Auxiliary lane	A lane in addition to, and placed adjacent to, a through lane intended for a specific manoeuvre such as turning, merging, diverging, weaving, and for slow vehicles, but not parking.
Back slope	Where the roadway is in cut, the slope between the drainage channel and the natural ground is referred to as a back slope.
Bikeway	A part of the right-of-way set aside for the preferential treatment of bicycle traffic, and is made up of one or more bicycle lanes.
Boulevard	A reserve which separates the roadway and sidewalk. It provides some protection to the pedestrian and can accommodate street accessories such as traffic signs and fire hydrants. It is a suitable location for underground utilities and may be used for illumination poles. It also provides an area for snow storage.
Brake reaction time	The time that elapses from the instant the driver decides to take remedial action, to the instant that remedial action begins (contacts brake pedal).
Braking distance	The distance travelled from the instant that braking begins to the instant the vehicle comes to a stop.
Broken back curve	An arrangement of curves in which a short tangent separates two curves in the same direction.
Bullnose	Location where edge of highway and edge of ramp meet each other. Bullnose may include or exclude curb and gutter.
Channelization	The separation of traffic flow into positive paths, by means of traffic markings and islands.
Collector lanes	Those lanes of an express/collector system separated from express lanes by an outer separation.
Crest vertical curve	A vertical curve having a convex shape in profile viewed from above.
Cross section	The transverse profile of a road.

Cross fall (cross slope)	The average grade between edges of a cross section element.
Crosswalk	Any part of a roadway specifically intended for pedestrian crossing, and indicated so by signs, lines or other markings.
Crown	The highest break point of the surface of a roadway in cross section.
Cul-de-sac	A road open at one end only.
Curb	A member with a vertical or sloping face along the edge of a lane or shoulder strengthening or protecting the edge or clearly defining the edge.
Curb and gutter	Curb and gutter is placed adjacent to an outside lane or shoulder and is intended to control and conduct storm-water and also provides delineation for traffic. In some instances, curb is introduced without a gutter.
Curve to spiral (CS)	The point of alignment change from circular curve to spiral curve, in the direction of stationing.
Curvilinear alignment	An alignment in which the majority of its length is circular and spiral curve.
Cut	A roadway located below natural ground elevation is said to be in cut.
Cut side slope	Where the roadway is in cut, the slope between the road-way and drainage channel is referred to as a cut side slope.
Deceleration lane	An auxiliary lane to enable a vehicle exiting from a roadway to reduce speed after it has left the through traffic lanes as applied at channelized intersections, or as speed-change lane at interchanges.
Decision sight distance	The distance required for a driver to detect an information source or hazard which is difficult to perceive in a roadway environment that might be visually cluttered, recognize the hazard or its potential threat, select appropriate action, and complete the manoeuvre safely and efficiently.
Deflection angle	The angle between a line and the projection of the preceding line.
Design speed	A speed selected for purposes of design and correlation of the geometric features of a road and is a measure of the quality of design offered by the road. It is the highest continuous speed at which individual vehicles can travel with safety on a road when weather conditions are favourable and traffic density is so low that the safe speed is determined by the geometric features of the road.
Design hour volume (DHV)	Volume used for design.
Drainage channel	A drainage channel is placed adjacent to an outside lane or shoulder and is intended to control and conduct storm-water runoff. A shallow drainage channel is sometimes referred to as a swale.
Entrance	The general area where turning roadway traffic enters the main roadway.
Entrance terminal	That part of an entrance comprised of acceleration lanes or speed change lanes including the ramp proper up to the ramp controlling curve.
Exit	The general area where turning roadway traffic departs from the main roadway.
Exit terminal	That part of an exit comprised of deceleration lanes or speed change lanes, including the ramp proper up to the ramp controlling curve.
Express/collector system	A freeway having an arrangement of four roadways adjacent to each other in which two roadways carry traffic in one direction and two in the other.

Express lanes	Those lanes of an express collector system separated by a median.
Expressway	A divided arterial road for through traffic with full or partial control of access and with some interchanges.
Fill	A roadway located above the natural ground elevation is said to be in fill.
Fill side slope	Where the roadway is in fill, the slope between the roadway and the natural ground is referred to as the fill side slope or sometimes the fill slope.
Flexible barrier	A form of longitudinal barrier that is intended to redirect an errant vehicle by rail tension, usually through a system of cables installed in tension.
Four-lane road	A road that provides two through lanes of traffic in each direction.
Freeway	A fully controlled access road limited to through traffic, with access through interchanges.
Friction factor	The coefficient of friction between tire and roadway, measured either longitudinally or laterally.
Frontage road	A road contiguous to a through road so designed as to intercept, collect and distribute traffic desiring to cross, enter or leave the through road and to furnish access to property.
Geometric design	The selection of the visible dimensions of the elements of a road.
Gore area	Area between edge of highway, edge of ramp and bullnose.
Gradient (Grade)	The rate of rise or fall with respect to the horizontal distance; usually expressed as a percentage.
Gravel road	A road that has a driving surface consisting of granular material.
Guiderail (Guardrail)	A longitudinal barrier of the general form of concrete, IBC barrier, steel beam or of posts and rail.
Gutter	A paved shallow waterway provided for carrying surface drainage.
Hazard	Any obstacle or other feature such as an embankment, or a body of water of depth greater than 1 m which, without protection, is likely to cause significant injury to the occupants of a vehicle encountering it.
Highway	Synonymous with through-road.
Horizontal alignment	The configuration of a road or roadway as seen in plan, consisting of tangents,lengths of circular curve, and lengths of spiral or transition curves.
Horizontal curve	A circular curve in plan to provide for change of direction.
Independent alignment	A divided highway in which each roadway is designed independently both in horizontal and vertical alignments, to take advantage of topographical features.
Interchange	A grade-separated intersection with one or more turning roadways for travel between the through roads.
Intersection (At-Grade)	The general area where two or more roads join or cross, within which are included the roadway and roadside facilities for traffic movements.
Island	A defined area between traffic lanes for control of vehicle movements or for pedestrian refuge and the location of traffic control devices.
Lane (Traffic Lane)	A part of the travelled way intended for the movement of a single line of vehicles.
Local road	A road intended to provide access to development only.

Longitudinal barrier	A barrier placed adjacent to a roadway, intended to contain a vehicle leaving the normal travel path, by re-directing it.
Low volume road	A road with average daily traffic of 200 vehicles per day or less, and whose service functions are oriented toward rural road systems, roads to or within isolated communities, recreation roads and resource development roads.
Median	The area that laterally separates traffic lanes carrying traffic in opposite directions. A median is described as flush, raised or depressed, referring to the general elevation of the median in relation to the adjacent edges of traffic lanes. The terms wide and narrow are often used to distinguish different types of median. A wide median generally refers to depressed medians sufficiently wide to drain the base and subbase into a median drainage channel. Flush and raised medians are usually narrow medians.
Median barrier	A longitudinal barrier placed in the median to prevent a vehicle from crossing the median and encountering oncoming traffic or to protect a vehicle form a fixed object in the median.
Minimum passing sight distance	The least visible distance required by a driver in order to make a passing manoeuvre safely, based on a given set of circumstances.
Minimum stopping sight distance	The least stopping sight distance required by a driver to come to a stop under prevailing vehicle, pavement and climatic conditions.
Multi-lane roads	Roads having more than two through lanes of traffic in each direction. In the Traffic and Capacity chapter Multilane refers to four lanes or more.
Obstacle	Any fixed object which is likely to cause significant injury to occupants of a vehicle encountering it.
One-lane one-way road	A road with one lane that carries one-directional traffic.
One-lane two-way road	A road that provides sufficient roadway width for the safe passing of opposing vehicles.
Outer separation	A reserve on freeways (including shoulders) between lanes carrying traffic in the same direction.
Overpass	A grade separation in which the subject road passes over an intersecting road or railway.
Passing opportunity sight distance	The distance ahead that must be visible to a driver to initiate a passing manoeuvre safely.
Passing sight distance	The distance ahead visible to the driver available to complete a passing manoeuvre.
Perception time	The time that elapses from the instant that a driver observes an object for which it is necessary to stop, until the instant that he decides to take remedial action.
Ramp	A turning roadway to permit the movement of traffic from one highway to another.
Reverse curve	Two curves, curving in opposite directions from a common point.
Right-of-way	The area of land acquired for or devoted to the provision of a road.
Rigid barrier	A form of longitudinal barrier that is intended to redirect an errant vehicle with minimum deflection in the barrier system and usually consists of a continuous concrete mass.
Road	The entire right-of-way comprising a common or public thoroughfare, including a highway, street, bridge and any other structure incidental thereto.
Roadside barrier	A longitudinal barrier placed adjacent to the right or left edge of a roadway, to prevent a vehicle leaving the roadway from encountering a hazard.
Roadway	That part of the road that is improved, designed or ordinarily used for the passage of vehicular traffic, inclusive of the shoulder.

Rounding	Width between edge of shoulder and cut or fill slope.
Rotary	A channelized intersection in which traffic moves counterclockwise around a centre island of sufficient size to induce weaving movements instead of direct crossings, sometimes referred to as a traffic circle.
Runout length	The distance parallel to the roadway, measured from the object to the point of vehicle encroachment. This distance varies with design speed and traffic volume.
Safety zone	An area officially established within a roadway for the exclusive use of pedestrians, protected or so indicated as to be plainly visible.
Sag vertical curve	A vertical curve having a concave shape in profile viewed from above.
Semi-rigid barrier	A form of longitudinal barrier that is intended to redirect an errant vehicle by a system of steel beam action to adjacent posts.
Service road	Same as frontage road but not necessarily contiguous with the through road.
Shoulder	Areas of pavement, gravel or hard surface placed adjacent to through or auxiliary lanes. They are intended for emergency stopping and travel by emergency vehicles only. They also provide structural support for the pavement.
Sidewalk	A travelled way intended for pedestrian use, following an alignment generally parallel to that of the adjacent roadway.
Sight distance (at intersections)	The distance along intersecting roads, resulting in a sight triangle, thereby providing a sight line to approaching vehicles. The intersection sight distance is adequate when a driver has an unobstructed view of the entire intersection and sufficient lengths of the intersecting roadway to avoid collision.
Simple open throat intersection	A simple or unchannelized intersection where additional area of pavement may be provided for turning of large vehicles.
Speed change lane	A deceleration or acceleration lane.
Spiral parameter (A)	"A" designates the sharpness of the spiral. It is a measure of the flatness of the spiral, the larger the parameter, the flatter the spiral.
Spiral to curve (SC)	The point of change from spiral curve to circular curve, in the direction of stationing.
Spiral to tangent (ST)	The point of change from spiral curve to tangent, in the direction of stationing.
Stopping distance	The distance travelled by a vehicle from the instant the driver decides to take remedial action, to the instant the vehicle comes to a stop (total of reaction and braking distances).
Stopping sight distance	The distance between a vehicle and an object, for which the driver decides to stop, to the instant the vehicle begins to comes into view (total of perception reaction and braking distances).
Stop block	Pavement marking to indicate where vehicles are required to stop for a traffic control device.
Summer average daily traffic (SADT)	The average 24-hour, two-way traffic for the period July 1st to August 31 st including weekends.
Superelevation	The gradient measured at right angles to the centre line across the roadway on a curve, from the inside to the outside edge.
Swale	A shallow drainage channel.
Tangent to spiral (TS)	The point of alignment change from tangent to spiral curve, in the direction of stationing.

Tangent runout	The length of road needed to accomplish the change in cross slope from a normal cross-section to a section with the adverse crown removed.
Through lane	A lane intended for through traffic movement.
Traffic barrier/Barrier	Traffic barriers are placed adjacent to a roadway to protect traffic from hazardous objects either fixed or moving (other traffic). Barriers placed in a median are referred to as median barriers and may be placed in flush, raised or depressed medians.
Transition (spiral) curve	A curve whose radius continuously changes.
Travelled way	That part of a roadway intended for vehicular use excluding shoulders. It may have a variety of surfaces but is most commonly hard surfaced with asphalt or concrete or gravel surfaced.
Turning roadway	A separate roadway or ramp to accommodate turning traffic at the intersection or interchange of two roads.
Two-lane road	A road that provides for one lane of through traffic in each direction.
Underpass	A grade separation in which the subject road passes under a highway or railway.
Vertical alignment	The configuration of a road or roadway as seen in longitudinal section, consisting of tangents and parabolic curves.
Vertical curvature (K)	The horizontal distance along a parabolic curve required to effect a one percent change in gradient.
Vertical curve	A parabolic curve on the longitudinal profile or in a vertical plane of a road to provide for change of gradient.
Warrant	A criterion that identifies a potential need or the justification for an addition to the highway such as traffic signals, traffic barrier, truck climbing lanes, passing lanes, left turn lanes etc.
Weaving section	A section of roadway between an entrance and an exit, such that the frequency of lane changing exceeds that for open highway condition.

CHAPTER A

HIGHWAY

CLASSIFICATION

CHAPTER A
HIGHWAY CLASSIFICATION

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HIGHWAY CLASSIFICATION

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A.1 INTRODUCTION

Highway systems are organized into a number of different operational divisions, functional classifications and geometric types for purposes of administration, planning and design. These classification systems provide a basis for communication among planners, designers, administrators and the public.

The primary reasons for road classification are to:

(1) establish logical integrated systems composed of all roads which, because of their service, should be administered by the level of government with the greatest basic interest.

(2) group roads that require the same level and quality of construction, maintenance and traffic service.

(3) establish a basis for funding requirements, desirable service standards and system expansion.

Many classification systems are used in Ontario. Four frequently encountered classification systems are described in the following section. The remainder of this chapter deals with the **Functional Classification System** which is the predominant method of classification for highway planning and design.

A.2 HIGHWAY CLASSIFICATION SYSTEMS

The Ministry has four widely used classification and administrative systems.

A.2.1 THE PUBLIC TRANSPORTATION AND HIGHWAY IMPROVEMENT ACT

The 'Public Transportation and Highway Improvement Act' legally defines the administrative and financial responsibilities of the provincial and municipal governments with regard to all roads, rapid transit systems and public transportation facilities in the Province of Ontario. The Public Transportation and Highway Improvement Act classifies all roads and highways into 13 separate administrative groups.

These are:

1. King's Highways (including "Intersecting Highways", "Crossing Highways", and "Connecting Link Extensions")
2. Controlled Access Highways
3. Secondary Highways
4. Tertiary Roads
5. Resource Roads
6. Industrial Roads
7. County Roads
8. Suburban Roads
9. Township Roads
10. City, Town and Village Roads
11. District, Metropolitan and Regional Municipal Roads
12. Development Roads
13. Roads in Territory without Municipal Organization.

A.2.2 PROVINCIAL HIGHWAY ACCESS CONTROLS

For policy guidelines dealing with land access control on Provincial Highways the Ministry has grouped the Provincial Highway System into five (5) classes:

1. Class I - Freeways and Expressways
2. Class II - Staged Expressways and Freeways
3. Class III - Special Controlled Access Highways
4. Class IV - Major Highways
5. Class V - Minor Highways

A.2.3 HIGHWAY INVENTORY MANAGEMENT SYSTEM

The Ministry has established a Highway Inventory database for the effective planning and management of roads in the Province not under municipal jurisdiction. This inventory generates material for the policy decision makers regarding funding requirements, desirable service standards, and system expansion. The Provincial highways are divided into:

1. The Desirable King's Highway System
2. The Desirable Secondary Highway System
3. Transfer Candidates

based on a system of "quantity standards" which relates to the service provided by the highway.

A.2.4 FUNCTIONAL CLASSIFICATION SYSTEM

Functional classification is the predominant method of grouping highways for transportation planning and design purposes. The Ontario Provincial Highways' "Future Perspective", the "Highway Inventory" process and the "System Management Plan" recognize the Functional Classification System as the major divisional unit for highway planning purposes. The objective of functional classification is to group highways, roads and streets into connected systems, having similar functions, purposes and importance in the highway network. Functional classification also differentiates between roads on the basis of land service, traffic service and traffic use.

A.3 FUNCTIONAL CLASSIFICATION SYSTEM COMPARED TO OTHER SYSTEMS

The other systems used by the Ministry were briefly described in Section A.2. They are, at first glance, dissimilar to the Functional Classification System. However, closer scrutiny reveals that similarities do exist. While no exact comparison is practical, Table A3-1 illustrates how each system may be compared to the Functional Classification System.

Table A3-1
CLASSIFICATION SYSTEMS COMPARED

FUNCTIONAL CLASSIFICATION SYSTEM	PUBLIC TRANSPORTATION AND HIGHWAY IMPROVEMENT ACT	PROVINCIAL HIGHWAY ACCESS CONTROLS	HIGHWAY INVENTORY MANAGEMENT SYSTEM
Freeway	Controlled Access Highway	Class I, II	Desirable King's Highway
Arterial	Controlled Access Highway King's Highway Secondary Highway	Class III, IV	Desirable King's Highway Desirable Secondary Highway
Collector	King's Highways Secondary Highway	Class V	Desirable King's Highway Desirable Secondary Highway
Local	Secondary Highway Tertiary Road Resource Road Development Road Roads in Territory without Municipal Organization remaining roads in this classification are part of the Municipal Road System	Class V	Desirable Secondary Highway Transfer Candidates

A.4 CLASSIFICATION SELECTION FOR HIGHWAY PLANNING AND DESIGN

The Functional Classification concept of grouping highways for transportation planning and design purposes is an important one for highway planners and designers. Although some geometric design criteria could be selected without first determining the functional classification, planners and designers should always keep in mind the overall purpose that the highway is intended to serve. Application of this concept is consistent with a systematic approach to highway planning and design.

The design standard presented in subsequent chapters of this manual cannot logically be used until the highway classification has been determined. The integration of functional classification in highway design will assist in unifying the planning and design process and is essential for the Provincial Highways' "System Management Plan."

A.5 THE FUNCTIONAL CLASSIFICATION SYSTEM

A.5.1 PURPOSE OF CLASSIFICATION

The main considerations for road classification systems are the travel demands of the public, land service based on existing and expected future land use and the overall continuity of the highway network.

Road systems are composed of a variety of road types performing two basic services:

1. to provide mobility by facilitating vehicle travel between points of origin and destination, and

2. to provide land access.

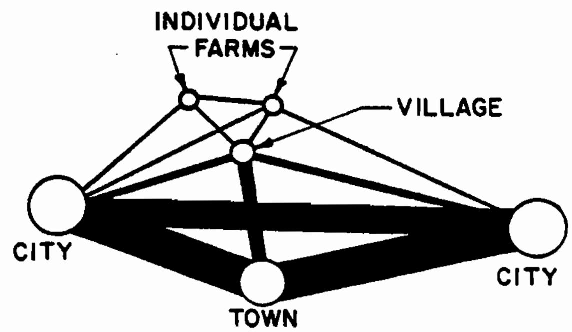
Access is a fixed requirement, necessary at both ends of any trip. Mobility is provided at varying levels of service along the trip route. Level of service can incorporate a number of qualitative descriptive elements such as riding comfort and freedom from speed changes, but the basic quantifiable factors are operating speed and trip travel time.

A.5.2 HIERARCHY OF TRAVEL AND FUNCTIONAL RELATIONSHIP

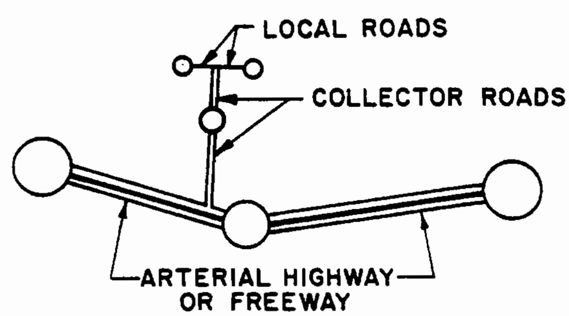
Most travel involves movement through a network of roads. This travel can be categorized by functional classification. A schematic illustration of this concept is shown in Figure A5-1.

The upper diagram shows the desired lines of travel between trip origins and destinations. The relative size of circles and width of lines represent the size of the generating or attracting power of the place, and the relative amounts of travel demand, respectively. Since it is impractical to provide direct line connections for every desired line, trips are channelled on a limited road network in a logical and efficient manner, as shown in the lower diagram of Figure A5-1. The heavy travel movements are directly served or nearly so; while the smaller travel movements are channelled along somewhat indirect paths. The facilities shown in the diagram are labelled local roads, collector roads and arterial highways or freeways. These are terms which describe their functional relationship.

The lower diagram of Figure A5-1 illustrates the concept of traffic channelization which leads not only to a hierarchy of functional classifications but also to a parallel hierarchy of relative travel distances. That is, trip lengths normally increase from local to collector to arterial. This hierarchy of travel distances is also related to the access and mobility requirements.



(a) Desire lines of travel



(b) Road network provided

Figure A5-1
Hierarchy of travel - trip channelization

A.5.3 ACCESS AND MOBILITY

Access and mobility are two major considerations in the functional classification of road systems. The conflicts between serving through movement and providing access to various trip origins and destinations necessitates the hierarchy of functional types.

Local facilities are normally short distance roads which emphasize the land access function. Arterials and freeways are normally long distance roads providing a high level of mobility for through movement with the freeways striving for optimum mobility. Collectors offer a balanced service for both functions. This concept is illustrated in Figure A5-2.

PROPORTION OF SERVICE

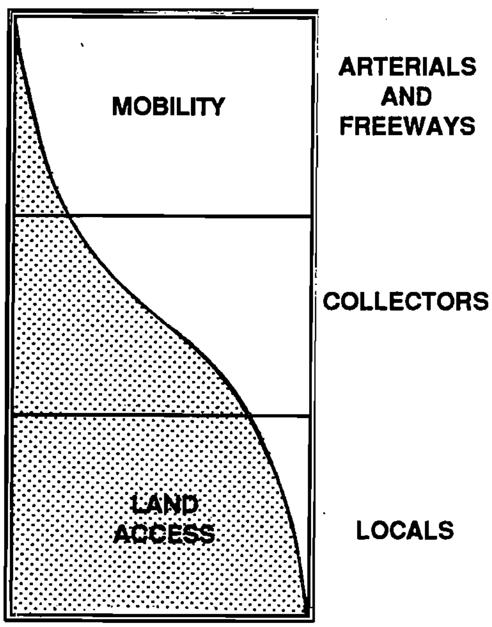


Figure A5-2
Traffic Mobility and Land Access

A.5.4 MAJOR OBJECTIVES

The functional classification system presented here performs the following objectives:

1. categorizes different roads on the basis of the service provided to the traffic mobility and land access,
2. categorizes different roads according to required geometric design standards,
3. relates to major jurisdictional and road classification systems presently in use.

A.5.5 MAJOR DIVISIONS

The functional classification system has eight major divisions as shown below in Table A5-1.

Table A5-1
HIGHWAY CLASSIFICATION - MAJOR DIVISIONS

RURAL	URBAN
Freeway	Freeway
Arterial	Arterial
Collector	Collector
Local	Local

The local, collector, arterial and freeway categories distinguish the functional characteristics. Although freeways perform a major arterial function, they are separated from arterials in the classification system because of differences in service function characteristics and geometric design standards.

The terms rural and urban refer to the predominant characteristics of the adjacent land use and not to jurisdictional boundaries. Rural and urban areas have fundamentally different characteristics with regard to density and types of land use, density of street and highway networks, nature of travel patterns, and the way in which all of these elements are related. Industrial, commercial and residential areas may dictate quite different geometric requirements. The potential social impacts on a community may warrant design considerations such as deviations in a corridor alignment, depressed sections, extra buffer strips and multiple use of land.

As a design guide, an urban environment may be assumed to exist where 50% or more of a road section over a distance of not less than 100 m is occupied by buildings, and the development is adjacent to the road and relies upon access to the road. This access may be limited to controlled locations as with reverse frontage service roads or urban freeways.

A.5.6 DESIGN CRITERIA

The eight major functional classification divisions are further subdivided for design purposes, because different roadways providing the same service may require quite different design criteria. These design criteria control the features that affect the overall 'quality' or level of service of travel on the road. Two design criteria which significantly affect the value of all other geometric design parameters are:

- divided vs. undivided highway
- design speed

A.5.6.1 Divided vs. Undivided

Divided roads are generally provided where traffic mobility is more important than land access. A divided road limits the number of opposing vehicle conflict points to controlled locations. The resultant road user benefits are improvements to level of service, safety and driver comfort.

When opposing traffic lanes are divided, the lateral separation is called a median. Medians are desirable when roads are expected to operate with significant volumes of traffic travelling at high speeds.

Normal applications include freeways and arterials.

Table A5-2 illustrates the application of divided and undivided criteria in the Functional Classification System.

A.5.6.2 Design Speed

Speed is one of the primary factors that a traveller considers when selecting alternate routes or transportation modes. The value of a transportation facility in carrying people and goods is judged by its convenience and economy, which are factors directly related to travel speed.

The aim in design of any public roadway is to satisfy the demands for service in the safest and most economical manner, with particular attention to speed demands. Provision should be made in design for an operating speed that will satisfy nearly all drivers. The operating speed of vehicles on a road or highway depends, in addition to the drivers' desired speed of travel and the capabilities of their vehicles, upon four general conditions:

- speed limitations (either legal or because of control devices)
- climatic conditions
- the presence of other vehicles
- the physical characteristics of the highway and its adjacent land use.

Although any one of these may govern, the effects of these conditions are usually combined.

Speed limitations are consciously introduced for reasons of safety and economy, traffic control and government regulatory policies.

Except for regular road maintenance operations, such as snow and ice removal, there is little control over climatic or weather conditions. However, the location of the highway and its cross-sectional arrangement can be arranged to minimize adverse weather effects.

With respect to traffic, roads can be designed to ensure reasonable operating speeds by appropriately controlling the volume/capacity (v/c) ratio. This is accomplished largely through the provision of the proper number and arrangement of lanes. This does not, however, entail an independent design procedure. The longitudinal characteristics of the highway are interrelated with the cross-sectional features through volume-capacity and design speed considerations. Thus, the standard to which a highway is designed is a primary determinant of operating conditions and speed of traffic on it.

This leads to the first factor, the physical characteristics of the highway, or the quality of geometry and, with it, the concept of "design speed".

Design speed has been defined as:

> "a speed used for the design and correlation of the physical features of a highway that influence vehicle operation", and as
>
> "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favourable that the design features of the highway govern".
>
> The "favourable conditions" refer to (a) good weather - clear, bright, dry and (b) low volume traffic on the highway, providing complete operational freedom.

Conventionally, the first stage of the geometric design process is to select a design speed. This "proposed" design speed should be a logical one with respect to the topography, the adjacent land use, and the type of highway. Every effort should be made to use as high a design speed as practicable to attain the desired degree of safety, mobility, and efficiency. Once selected, all of the pertinent features of the highway should be related to the design speed to obtain a balanced design. Desirable design values should be used where feasible, but in view of the numerous constraints often encountered, acceptable values are recognized and used. Some features, such as curvature, superelevation, and sight distance are directly related to, and vary appreciably with, design speed. Other features, such as widths of pavements and shoulders and clearances to walls and rails, are less directly related to design speed, but they affect vehicle operating speed, and higher standards should be accorded these features for the higher design speeds. Thus, when a change is made in design speed, nearly all design elements of the highway are subject to change.

Table A5-2 illustrates the application of design speed in the Functional Classification System.

A.5.6.3 Design Consistency

Since all design elements are thus interrelated, it is important to maintain design consistency when designing or redesigning a highway. Design consistency exists when the geometric features of the highway are consistent with the operational characteristics as perceived by the driver.

The traditional approach to achieving design consistency has been through the application of the design speed process. Once selected, the design speed is used to determine values for the geometric design elements from appropriate design tables.

Direct application of this procedure does not necessarily guarantee design consistency however. There are several limitations of the design speed concept that should be considered during design:

1. Selection of standards permitted by a specified design speed does not necessarily ensure a consistent alignment design.

   Design speed is significant only when physical roadway characteristics limit the safe speed of travel. The design speed specifies minimum values for the geometric design elements. Above minimum values are normally recommended wherever terrain and economy permit. Thus, a road can be designed with a constant design speed as conceived by the designer, yet have considerable variation in speed standards and to a driver, appear to have a wide variation in design standard.

   To maintain a consistent standard, the design speed of curves within a highway section should be uniform. Guidelines for curvature speed standards are presented in Section A.5.7.

2. Selection of design values permitted by design speed does not necessarily ensure compatibility between the standards for combinations of design elements.

   Minimum acceptable standards for isolated design elements do not provide the same degree of safety when the elements occur in combination.

For additional information on this aspect of alignment design refer to Chapter C, ALIGNMENT.

3. Vehicle operating speeds are not necessarily synonymous with design speed.

   Drivers normally adjust speed according to their desired speed of travel, posted speed and to the perceived alignment hazards. As presented above, the speed standard and hence, the perception of hazard presented by the alignment may vary along a road designed with a constant design speed. The speed adopted by a driver tends to vary accordingly and may often be in excess of the design speed. In addition, different alignment elements may have quite different levels of perceived hazard. Entering a horizontal curve at excessive speed will almost certainly result in a loss of control situation, so drivers adjust their speed accordingly. However, the possibility of a curtailed sight distance concealing a hazard is considered as a remote occurrence. Drivers do not generally adjust their speed to a level commensurate with a sight distance restrictions.

   It is important therefore, to:

   - retain a uniform alignment standard, and
   - avoid the application of minimum acceptable standards for vertical curves in the vicinity of intersections or entrances which could generate turning or stopped traffic.

4. Design inconsistencies can be created during highway reconstruction by upgrading selected design features while retaining others.

   An example of this is upgrading a highway cross-section without improving a deficient alignment. Often lane widths and shoulders are widened on an old two-lane highway to accomplish an "economical" improvement. The new wider cross-section combined with the deficient alignment conveys a conflicting message to the driver. The expected quality of operation based on the high standard cross-section width may be quite different from the actual limitations imposed on the operation by the lower standard alignment. The initial driver response to this incongruency could be delayed or incorrect, creating a potentially unsafe condition.

A.5.7 DESIGN SPEED SELECTION

Many factors influence and constrain the selection of the appropriate design speed for a given highway facility, which include:

- traffic conditions, such as volumes, composition and trip length
- character of terrain
- socio-economic-political characteristics of the area, i.e. population density and land development and travel habits of the local residents
- environmental quality and aesthetics
- economics

Application of these criteria applies only to the selection of a specific design speed within the logical range of values pertinent to the classification type selected. The ranges for each classification are illustrated in Table A5-2.

Traffic volumes are instrumental in the selection of the appropriate classification from the eight basic types, as well as in the selection of road cross-sectional features and intersection/interchange design which affect the capacity and level of service.

The effects of terrain types, socio-economic characteristics, environment and economics are not immediately obvious.

The typical driver can recognize or sense a logical operating speed for a given highway based on knowledge of the system, appraisal of the ruggedness of the terrain, and the extent, density and size of development. Based on this judgement, the driver will adjust speed to be consistent with the conditions expected to be encountered. The driver's initial response is to react to the anticipated situation rather than to the actual situation. In most instances, the two are similar enough that no problems are created. When the initial response is incorrect; operation and safety may be severely affected.

Design speed should be chosen to be consistent with the speed a driver is likely to expect. Where a difficult condition is obvious, drivers are more inclined to accept lower speed operation than where there is no apparent reason for it.

Other things being equal, it follows that a highway in level or rolling terrain justifies a higher design speed than one in mountainous terrain, and a highway located in a rural environment calls for a higher design speed than one situated in an urban area.

A highway carrying a large volume of traffic may justify a higher design speed than a less important facility in similar topography, particularly where the savings in vehicle operation and other operating costs are sufficient to offset the increased costs of right-of-way and construction. A low design speed, however, should not automatically be assumed for a secondary highway where the topography is such that drivers are likely to travel at high speeds. Drivers do not adjust their speeds to the importance of the highway but to its physical limitations and the traffic thereon.

When the appropriate highway classification division is selected from Table A5-1. the design speed can be chosen from the range of values in Table A5-2.

When designing a substantial length of highway, it is desirable, although it may not be feasible, to assume a constant design speed. Changes in terrain and other physical controls may dictate a change in design speed on certain sections. Each section, however, should be of relatively long length, compatible with the general terrain or development through which the highway passes. The justification for introducing a reduced design speed should be obvious to the driver. Moreover, the introduction of a lower or higher design speed should not be effected abruptly but over sufficient distance to permit drivers to change speed gradually before reaching the section of highway with the different design speed.

Differences in design speed from one segment to another should not be more than 20 km/h. Even so, drivers may not perceive the slower condition ahead, for which they should be warned well in advance. A transition section allowing for speed reductions, as from 100 to 90 to 80 km/h should be provided. Thus, the changing condition should comprise extra long (anticipatory) sight distances, speed-zone signs, curve speed signs, and so on.

Design speed should be greater than or equal to the legal posted speed.

Generally the desirable practice of selecting the design speed for new construction and reconstruction is

- 20 km/h greater than the proposed legal speed,unless circumstances warrant a reduction.

A design speed equal to the maximum posted speed is accepted where warranted by such factors as low traffic volumes, rugged terrain and economic considerations. This practice would be more appropriate for minor collector and local roads. A design speed equal to the legal posted speed is the normal practice for Secondary Highways.

Where a highway section warrants only resurfacing to remove pavement structure deficiencies, the general practice is to limit construction costs by removing only critical deficiencies as identified by the accident and maintenance records. The existing alignment is generally retained. In these situations the proposed and the existing design speeds should be the same.

Commonly used design speeds are:

- 120 km/h for freeways
- 110 km/h for major arterials carrying long distance traffic, and all four-lane divided and undivided highways
- 100 km/h for all other arterials and collectors
- 80 km/h for local roads and secondary highways.

Horizontal and vertical alignment geometry should be consistent with the selected design speed. In practice, because of numerous constraints often encountered, minimum acceptable values for alignment standards are recognized and used.

Minimum acceptable standards are based on the allowable reduction in the design speed of isolated curves from the overall design speed of the highway.

The reduction should preferably be no greater than 10 km/h and never greater than 20 km/h.

Where higher than average accident rates can be attributed to geometric design deficiencies, corrective measures should be considered. Isolated deficiencies should be improved if signing alone proves to be ineffective and costs are acceptable.

Where a minor secondary highway has a generally substandard alignment and advisory and warning signs have proven ineffective, consider:

- where no improvements are warranted - reducing the legal posted speed to be consistent with the overall highway design speed; or
- where improvements are warranted - selecting a design speed and corresponding legal posted speed commensurate with the topography and with a realistic balance between improvement costs and user benefits.

The implications of employing substandard curvature are more fully explained in Chapter C - ALIGNMENT.

Design speeds have been established in 10 km/h increments, ranging from 40 km/h for local roads, to a maximum of 120 km/h for design of freeways. Maximum and minimum design speeds have been established for each major classification of highway. The resulting functional classification system is presented in Table A5-2.

Table A5-2
THE FUNCTIONAL CLASSIFICATION SYSTEM

RURAL				
Design Speed km/h	**Freeway**	**Arterial**	**Collector**	**Local**
120	RFD 120			
110	RFD 110	RAD 110 RAU 110		
100	RFD 100	RAD 100 RAU 100	RCU 100	
90		RAD 90 RAU 90	RCU 90	
80		RAD 80 RAU 80	RCU 80	RLU 80
70			RCU 70	RLU 70
60			RCU 60	RLU 60
50				RLU 50
URBAN				
120	UFD 120			
110	UFD 110	UAD 110 UAU 110		
100	UFD 100	UAD 100 UAU 100		
90	UFD 90	UAD 90 UAU 90	UCU 90	
80		UAD 80 UAU 80	UCU 80	
70			UCU 70	
60			UCU 60	ULU 60
50			UCU 50	ULU 50
40				ULU 40

Legend of Abbreviations

FIRST LETTER: R - Rural
U - Urban

SECOND LETTER: F - Freeway
A - Arterial
C - Collector
L - Local

THIRD LETTER: D - Divided
U - Undivided

NUMBER: Design Speed

Example:

RFD 120 - Rural Freeway Divided, Design Speed 120 km/h

A.5.8 CLASSIFICATION SELECTION

The first step in any road planning, design or administrative study is to define the function that the facility is to serve. This exercise will define important functional aspects such as the degree of mobility and land access to be provided by the facility. The level of service required to fulfil this function for the anticipated volume and composition of traffic provides a rational and cost-effective basis for the selection of design speed and geometric criteria.

The important characteristics to be considered are described below in the following sections.

A.5.8.1 Service Function

All roads provide service to traffic, access to land, or a combination of the two. Freeways and arterials provide for the movement of through traffic. Local roads are used almost exclusively for land access. Collectors provide a combined service.

A.5.8.2 Traffic Volume

High volumes of traffic are generally associated with freeways and arterials, while low volumes are associated with collectors and locals. However, the design hour volume (D.H.V.) and annual average daily traffic (A.A.D.T.) in the design year are only two factors to be considered in the classification. The volume range for each class is wide and overlaps that of other classifications. For more details of D.H.V. and A.A.D.T., refer to Chapter B, TRAFFIC AND CAPACITY.

A.5.8.3 Traffic Flow

The desired characteristics of traffic flow will, in general, determine the classification of a road. For example, roads primarily serving traffic movement such as freeways and arterials, are expected to have uninterrupted traffic flow characteristics. The flow on local roads and streets, which are designed to provide full land service, is necessarily restricted by traffic crossing, entering and leaving the roadway and, in particular, by parked vehicles. Pedestrian traffic, which may be controlled to varying degrees on locals, collectors and arterials, also has a significant effect on the traffic flow, particularly in urban areas.

A.5.8.4 Running Speed

The average running speed of traffic operating under off-peak volume conditions will vary on roads of the same classification depending on the condition of the pavement, intensity of adjacent land development, access to the roadway, vehicle types, geometrics and traffic flow controls. Running speeds generally increase from locals to collectors to arterials to freeways.

A.5.8.5 Vehicle Types

The proportion of buses, trucks, transports and passenger cars using a road is generally dependent upon the purposes of the road and is, therefore, related to the road classification. Local roads are generally used predominantly by passenger cars and small trucks, with a small percentage of large trucks. Freeways and arterials generally carry a higher proportion of commercial vehicles than collectors and locals.

A.5.8.6 Percentage of Total System

There is a general relationship between the total length of a given road classification and the total length of the whole system in either a rural or an urban area. There will normally be a very high percentage of local road length and progressively lower percentages of collectors, arterials and freeways.

A.5.8.7 Connections

In a road system, the connections between different road classifications as shown in table A5-3 are desirable.

TABLE A5-3
DESIRABLE CLASSIFICATION CONNECTIONS

Classification	Connects to (Classification)
Freeway	Freeway Arterial
Arterial	Freeway Arterial Collector
Collector	Arterial Collector Local
Local	Collector Local

It is prefereable to minimize the interconnection of locals with arterials and of collectors with freeways. Locals should rarely connect with freeways.

A.5.9 DESCRIPTION OF CLASSIFICATIONS

The principal characteristics of each of the eight basic classifications are described in the following sections.

A.5.9.1 Rural Freeways

· Service Function

Rural freeways are built to accommodate the movement of large volumes of traffic at high speeds under free-flow conditions. Rural freeways connect the larger cities, industrial concentrations and recreational areas. They also serve as the major highway routes through intensely developed areas and serve larger international and interprovincial travel movements. The need for unrestricted traffic movement on these facilities justifies the elimination of direct property access. Ultimate development will require grade separation at all crossing roads.

· Traffic Volume

A freeway might be required when traffic volumes are 10,000 A.A.D.T. and greater. While a relatively high volume is generally a prerequisite for a freeway, the need for high speed movement under free flow conditions must also be present.

· Traffic Flow

On rural freeways, traffic flow should be uninterrupted and unrestricted. Opposing traffic lanes must be separated. Access should be controlled with grade separations or interchanges provided at all road, rail or pedestrian crossings. Parking should be prohibited by regulation on all controlled access highways.

· Design Speed

Rural freeways are intended to move large volumes of traffic at high speeds under free-flow conditions. Design speeds should be chosen from the range of 100 - 120 km/h.

· Running Speed

The average running speed under most conditions is between 80 and 120 km/h on rural freeways.

· Vehicle Types

Rural freeways carry all types of vehicular traffic, heavy truck transports normally amount to between 20 and 30% of the total volume.

· Percentage of Total System

Rural freeways are constructed to connect large cities or heavily developed areas. The total length of these roads is normally less than 5% of the total length of rural roads.

· Connections

Rural freeways should interchange traffic only with other freeways, arterials and collectors. The interconnection of freeways with local roads is not a desirable practice.

· Stage Construction

All rural freeways should normally be planned and designed as multi-lane, divided, controlled access facilities even though they may be developed by stage construction. In the plans for each stage of development, provision should be made to adapt each stage to the next or ultimate stage. The transition should be made with minimum waste of the existing plant and minimum interference to traffic.

Stage construction is dealt with in more detail in Chapter D, Section D.1.2 and Section D.6.

A.5.9.2 Rural Arterial Roads

· Service Function

Rural arterial roads are intended to move large volumes of traffic at high speeds. The major distinction between this classification and the freeway classification is in the full control of access. Roads that have full control of access should normally be in the freeway classification. Rural arterial roads, together with freeways, serve as the major routes in a network connecting the major economic regions and centres of a province such as large cities, industrial concentrations, agricultural areas and recreational facilities. In areas where freeways are not warranted by the traffic service required, rural arterials are the highest type of road. Because arterials carry large traffic volumes moving at high speed, direct access to abutting lands may be restricted or even eliminated. This applies particularly in areas of intensive development and in undeveloped areas where the lack of other road service would encourage strip development.

· Traffic Volumes

The annual average daily traffic (AADT) volumes on rural arterials vary from 1000 to 20,000 vehicles. This wide range occurs because arterials in the sparsely populated regions have relatively low traffic volumes while providing an important inter-regional traffic service. Average daily traffic volumes of 20,000 vehicles occur on arterial highways in very densely populated areas.

· Traffic Flow

Rural arterial roads carry large traffic volumes at high speeds and should be designed for uninterrupted flow of traffic except at controlled intersections with crossing roads. However, intersections on high speed roads controlled by traffic signals or stop signs can be hazardous and should be avoided if possible. In some cases, grade separated interchanges are warranted by traffic volumes. Bicycle and pedestrian traffic is sometimes prohibited by regulation on controlled access arterial roads.

· Design Speed

Rural arterials perform a function similar to the rural freeways, but do not have full control of access. Lower design speeds, selected from the range of 80 to 110 km/h are appropriate.

· Running Speed

The average running speed is between 60 and 100 km/h. The higher values are found on those sections of rural arterial roads having some control of access.

· Vehicle Types

Rural arterial roads link the major regions of a province and serve all types of vehicles. Up to 20% of traffic is heavy trucks.

· Percentage of Total System

Rural arterial roads normally make up from 5 to 10% of the total rural road length depending upon the extent of rural freeway development in the area.

· Connections

Rural arterial roads connect with all other classifications of rural roads.

A.5.9.3 Rural Collector Roads

· Service Function

Rural collector roads collect traffic from local roads and feed it to arterials, or distribute it from arterials to locals. They generally form an integrated network throughout all developed areas of the province and provide direct traffic service for development such as tourist areas, mining areas and the small towns and villages. Rural collector roads have a land service function of equal importance to their traffic service function in that they directly serve the adjacent properties.

· Traffic Volumes

Average daily traffic volumes on rural collector roads may very from 200 to 10,000 vehicles depending on the population density.

· Traffic Flow

Traffic flow on rural collector roads is interrupted by stop conditions or signalized intersections with arterials or other collector roads. Traffic flow is also impeded by vehicles leaving and entering the highway directly from adjacent properties. Bicycle and pedestrian traffic is not restricted.

· Design Speed

Since rural collectors serve a dual role of traffic movement and land access, lower design speeds varying from 60 - 100 km/h are desirable.

· Running Speed

The average running speed on rural collector road varies between 60 and 90 km/h.

· Vehicle Types

Although rural collectors carry all types of vehicles, commercial traffic consists mainly of single unit trucks. In agricultural areas these are normally farm trucks carrying produce such as milk, feed and livestock. These trucks amount to as much as 30% of the volume of traffic using these roads. Few heavy transport trucks use this class of road except when the road serves mining and resource land uses.

· Percentage of Total System

Rural collector roads make up from 10 to 20% of the total length of rural roads.

· Connections

Rural collector roads connect with all other classifications of roads except freeways.

A.5.9.4 Rural Local Roads

· Service Function

The main function of rural local roads is to provide land access. The only traffic service function of a local road is to allow vehicles to reach the frontage of properties from the main highways. Development roads that serve natural resource areas may be considered as local roads until volumes and function justify reclassification.

Table A5-4
CHARACTERSTICS OF RURAL ROAD CLASSSIFICATIONS

FUNCTIONAL CLASSIFICATION	RURAL FREEWAYS	RURAL ARTERIALS	RURAL COLLECTORS	RURAL LOCALS
Traffic Service	optimum mobility	traffic movement primary consideration	traffic movement & land access equal importance	traffic movement secondary consideration
Land Service	no access	land access secondary consideration	traffic movement and land access equal importance	land access primary consideration
Range of Traffic Volume A.A.D.T	more than 10,000	1,000 - 20,000	200 - 10,000	not applicable
Traffic Flow	free flow	uninterrupted flow except at signals	interrupted flow	interrupted flow
Design Speed	100 - 120 km/h	80 - 110 km/h	60 - 100 km/h	60 - 80 km/h
Average Running Speed Off-peak Conditions	80 - 120 km/h	60 - 100 km/h	60 - 90 km/h	50 - 80 km/h
Vehicle Type	all types heavy trucks average 20 - 30%	all types up to 20% trucks	all types up to 30% trucks mostly single unit type	predominantly passenger cars and light to medium trucks and occasional heavy trucks
Percentage of Total Length	up to 5	5 - 10	10 - 20	75 approx.
Connects to	freeways arterials collectors	all classifications	all classifications	arterials collectors locals

· Traffic Volumes

Traffic volumes on rural local roads are generally low but could be several hundred vehicles per day, depending on the density of development along the sides of the road.

· Traffic Flow

Traffic flow on rural local roads is normally interrupted by stop conditions at all intersecting roads and is affected by traffic moving to and from adjacent properties. Pedestrian traffic is not restricted.

· Design Speed

Rural local roads primarily serve land access. Lower design speeds from 60 - 80 km/h are appropriate.

· Running Speed

Depending on the condition of the surface, the average running speed on rural local roads varies from 50 to 80 km/h.

· Vehicle Types

In agricultural areas, trucking is done by light and medium single unit vehicles with occasional heavy trucks. In mining and forestry areas, heavy units will predominate. The number of trucks depends upon the adjacent land use and ranges up to 50% of the total vehicular volume.

· Percentage of Total System

Local roads normally make up approx. 75% of the total length of rural roads.

· Connections

Rural local roads connect with collector and arterial roads.

A.5.9.5 Urban Freeways

· Service Function

Urban freeways are intended to accommodate heavy volumes of traffic moving at high speeds under free-flowing conditions. Urban freeways connect major points of traffic generation and may serve as urban extensions of principal rural highways. They are intended to service traffic between large residential areas, industrial or commercial concentrations and the central business district. To provide optimum mobility for through traffic, service to adjacent lands is completely eliminated. No parking, unloading of goods, or pedestrian traffic is permitted.

· Traffic Volume

Freeways may be required when traffic volumes reach 75,000 A.A.D.T.

· Traffic Flow

To move high volumes at high speeds, it is necessary to have uninterrupted flow conditions on urban freeways. These conditions can only be provided by grade-separated crossings and interchanges. Parking and pedestrian access is prohibited. Because of the difficulty of providing the desired service and operating conditions on existing streets, urban freeways are generally constructed on new alignments.

· Design Speed

Urban freeways, like their rural counterpart, should provide the highest standards of highway design. The range of design speeds varies from 80 - 120 km/h.

· Running Speed

The normal running speed under free-flow conditions varies between 60 and 110 km/h.

· Vehicle Types

Urban freeways are expected to carry all types of vehicles including a relatively high percentage of transport trucks, amounting up to 20% of the total volume. Only express bus service with no stop is permitted on urban freeways.

· Percentage of Total System

The total length of urban freeways is normally less than 10% of the total length of urban roads.

· Connections

Urban freeways are directly connected to intersecting or adjacent freeways and to most intersecting or adjacent arterial streets. Some direct connections to collector streets may be provided in the central business district.

· Stage Construction

All urban freeways should normally be planned and designed as multi-lane, divided, controlled access facilities even though they may be developed by stage construction. In the plans for each stage of development, provision should be made to adapt each stage to the next or ultimate stage. The transition should be made with minimum waste of the existing plant and minimum interference to traffic.

Stage construction is dealt with in more detail in Chapter D, Section D.1.2 and Section D.6.

A.5.9.6 Urban Arterials

· Service Function

Urban arterial streets are intended to carry large volumes of all types of traffic moving at medium to high speeds. These streets serve the major traffic flows between the principal areas of traffic generation and also connect to arterials and collectors. In urban areas without freeways, arterial streets provide the best quality of traffic service.

Urban arterial streets should limit the amount of direct private access to adjacent development. Desirably, this access should be confined to local and collector road connections, but utilizing such treatments as reverse road frontages, side road access and so on.

· Traffic Volumes

Urban arterial streets normally experience average daily traffic volumes of 5000 to 50,000. Urban arterials may be divided or undivided, depending on the amount of traffic to be served.

· Traffic Flow

The traffic flow is, desirably, uninterrupted except at signalized intersections and crosswalks. Where signals are closely spaced, they should be interconnected and synchronized to minimize the interference to through movements. Parking and unloading should be prohibited where they might affect through movement of traffic, particularly at rush hours. Pedestrians should be permitted to cross only at intersections or designated cross walks. Public transit loading stations should be designed to minimize interference with through traffic movements. Turning lanes may be provided at intersections.

· Design Speed

Urban arterials are intended to carry traffic at medium to high speeds. Design speeds should be selected from the 80 - 110 km/h range.

· Running Speed

Running speed under free-flow conditions normally ranges from 50 - 90 km/h with the higher values prevailing in suburban areas.

· Vehicle Types

All types of vehicular traffic use urban arterial streets. Trucks of all sizes may comprise as much as 20% of the total traffic volume. Both express and local buses are generally routed on arterial streets.

· Connections

Urban arterial streets connect with freeways, arterials and collectors.

· Percentage of Total System

The combined length of urban collectors and urban arterials may be as much as 30% of the total length of urban streets.

A.5.9.7 Urban Collectors

· Service Function

Urban collector streets provide both traffic service and land service. The traffic service function of this type of street is to carry traffic between local and arterial streets. Full access to adjacent properties is generally allowed on collectors.

· Traffic Volumes

The average daily traffic in the design year normally ranges between 1,000 and 20,000 vehicles. Collector streets can have more than two traffic lanes and may be divided.

· Traffic Flow

The traffic flow on urban collector streets, in and near the central business district, is interrupted frequently by signalized intersections. In residential areas, simpler forms of traffic control are generally used. There are few parking restrictions except during peak hours when traffic movement may be the most important consideration. There are generally no special pedestrian crossing restrictions, but special cross walks might be provided where traffic volumes are high. To improve traffic flow, particularly at peak hours, it is sometimes desirable to provide collector streets with bus bays or turning lanes similar to those provided on arterial streets.

· Design Speed

Urban collectors operate at low to medium speeds since traffic flow and land access are of equal importance. Design speeds may range from 60 - 90 km/h.

· Running Speed

The normal running speed under free-flow conditions varies between 40 and 70 km/h, with the higher value prevailing in urban fringe areas.

· Vehicles Types

In commercial and industrial areas, all types of vehicles use urban collector streets, including truck transports moving to and from arterials. In residential areas, collectors will carry a low percentage of trucks composed mainly of service vehicles.

· Percentage of Total System

The combined total length of arterial and collector streets may amount to 30% of hte total length of urban streets.

· Connections

Urban collector streets are connected to arterial and local streets, but connections to freeways are rarely found except in the central business district.

A.5.9.8 Urban Locals

· Service Function

The main function of local streets is to provide land access. Direct access is allowed to all abutting properties. Local streets are not intended to move large volumes of traffic. The local street primarily carries traffic with an origin or destination along its length. It is not intended to carry through traffic other than to immediately adjoining streets. Local streets may be residential or commercial. Residential developments may carry appreciably higher traffic volumes and may, therefore, be multi-lane but they are seldom divided.

· Traffic Flow

Local streets have stop, yield or signalized controls where they intersect more important streets. Parking may be prohibited or restricted to one side on narrow streets. Pedestrian traffic is unrestricted.

· Design Speed

Urban local streets primarily serve a land access function. Design speeds are necessarily low, ranging from 60 - 80 km/h.

· Running Speed

Running speed under free-flow conditions is generally 40-60 km/h.

· Vehicle Types

The type of vehicle using local streets will vary. Residential streets carry predominantly passenger vehicle and the occasional transport. Industrial streets carry a high percentage of trucks and transports. Bus operations rarely occur on residential streets except at a "turn-about".

· Percentage of Total System

The length of local streets amounts to about 70% of the total length of urban streets.

· Connections

Local streets connect to other local streets and collector streets. It might be necessary to have industrial or commercial local streets connect directly to arterials.

Table A5-5
CHARACTERISTICS OF URBAN ROAD CLASSIFICATIONS

FUNCTIONAL CLASSIFICATION	URBAN FREEWAYS	URBAN ARTERIALS	URBAN COLLECTORS	URBAN LOCALS
Traffic Service	optimum mobility	traffic movement primary consideration	traffic movement & land access equal importance	traffic movement secondary consideration
Land Service	no access	land access secondary consideration	traffic movement and land access equal importance	land access primary consideration
Range of Traffic Volume A.A.D.T	more than 75,000	5,000 - 50,000	1,000 - 20,000	not applicable
Traffic Flow	free flow	uninterrupted flow except at signals and cross walks	interrupted flow	interrupted flow
Design Speed	80 - 120 km/h	80 - 110 km/h	60 - 90 km/h	60 - 80 km/h
Average Running Speed Off-peak Conditions	60 - 110 km/h	50 - 90 km/h	40 - 70 km/h	40 - 60 km/h
Vehicle Type	all types up to 20% trucks	all types up to 20% trucks	all types	passenger and service vehicles
Percentage of Total Length	up to 10	up to 30	up to 30	70 approx.
Connects to	freeways arterials	freeways artetials collectors	arterials collectors locals	collectors locals

A.6 DESIRABLE VERSUS ACCEPTABLE DESIGN

Highway design includes the following:

- the selection of the functional classification
- the selection of the geometric design parameters which are dependent on the functional classification

The selection of the appropriate classification type is a complex decision process that is subject to a variety of imposed constraints and influencing factors. In most situations, this decision process will first isolate the appropriate major division, from Table A5-1, then narrow the selection to a range of possible design speeds from Table A5-2.

The highway design speed is closely related to the legal posted or operating speed. Research has established that for operational and safety considerations, the design speed should desirably be 20 km/h greater than the posted speed. An acceptable relation is one where the design speed equals the posted speed. The highway posted speed is generally a known parameter based on general policy decisions related to the overall hierarchy of the highway and road system.

The decision for desirable versus acceptable then, usually depends on the economics of achieving the additional margin of safety and capacity normally associated with the desirable design. Of course, a "desirable" design is always preferred over an "acceptable" design if economics are not a consideration. There are circumstances, however, where conditions warrant the use of the acceptable design speed. Acceptable design speeds may be appropriate where the highway is a minor link in the system. For these types of roads, operating speeds are in the low to medium range, and traffic volumes are expected to remain at low levels for the foreseeable future. Although the highway fulfils a provincial mandate in terms of public service, the low speeds and traffic volumes justify consideration of the "acceptable" design standard.

Every effort should be made to use the desirable standard on freeways, arterials and major collectors which are generally important links in the highway system. These roads normally carry significant traffic volumes at medium to high speeds. Considering the service life and function expected of these highways, desirable standards are the preferred alternative. As was discussed in Section A.5.6.2, Design Speed, achieving the desirable standard is not always practical or realistic when faced with terrain, urban development and economic factors.

The selection of the geometric design parameters is dependent on the functional classification. Some of these parameters have desirable and minimum values for each design speed. The same argument used for the selection of desirable versus acceptable design speeds also applies to the selection of the appropriate values for the geometric design parameters.

Whether design speed or design parameter is under debate, the highway planner/designer must never lose sight of the overall objective - to create a safe, efficient and economical transportation system. Several concepts for improved design have been developed which directly support this goal. These include design consistency and uniformity.

Design consistency has been defined as the condition which exists when the geometric features of the roadway are consistent with the operational characteristics as determined by what the driver expects of the roadway ahead and what he is willing to accept in terms of operational quality.

The concept of operational uniformity is similar in nature to that of consistency. It refers to a consistent arrangement of geometric features, exits and entrances that reinforce the driver's confidence or expectancy.

Consistency and uniformity of design standards place the driver in an environment which is fundamentally safer because it is more likely to compensate for the driving errors that unfortunately are inevitably made.

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B.1 INTRODUCTION

This chapter of the manual provides the designer with a set of procedures and techniques to analyze existing facilities and proposed designs for their ability to carry traffic. The analysis techniques cover a broad range of facilities and methods of traffic control.

Facilities are generally classified into two categories, uninterrupted flow and interrupted flow.

Uninterrupted flow roads have no fixed control devices such as traffic signals or "stop" signs. Traffic flow conditions are the product of interaction between vehicles and geometric factors of the roadway. Most rural highways and, rural and urban freeways fall into this category.

Interrupted flow roads , on the other hand, have fixed elements causing periodic interruptions to traffic flow. These include traffic signals and stop signs, and cause traffic to stop regardless of traffic flow.

The terms uninterrupted and interrupted describe the type of facility and not the quality of traffic flow.

The procedures in this chapter are derived from two primary sources: techniques developed by the Ministry and procedures taken from the Highway Capacity Manual Special Report 209, Transportation Research Board, 1985 (HCM 1985). This latter document is in imperial units, and for use in this chapter the procedures have been converted to metric using a hard conversion approach. The terminology of HCM 1985 has been adopted with the exception of two expressions:

- The word "flow" is used in place of the expression "flow rate" and "rate of flow".
- The term "ramp terminal" is used in place of "ramp junction" to be consistent with the terminology of Chapter F.

In other chapters of this manual, the term "multi-lane" is used to mean six lanes or more. In this chapter the term is used to refer to four lanes or more, to be consistent with HCM 1985.

"Transit" and "Pedestrians" are not covered in this chapter and for information on these subjects, reference may be made to the Highway Capacity Manual, 1985, Chapters 12 and 13.

B.2 TRAFFIC CHARACTERISTICS

B.2.1 Traffic Volumes

In the terminology of temporal variations, the unit of variation of the time period is smaller than the time period, illustrated in the Table B2-1.

Table B2-1
VARIATION OF TIME PERIOD

Time Period	Unit of Variation of the Time Period
Year	Season, Month, Week, Day
Week	Day
Day	Hour
Hour	15-minute period

From the Table B2-1, the following terminology is derived:

- "Hourly Variation" means within the day.
- "Daily Variation" means within the week or other time period greater than one week.
- "Weekly Variation" means within the year or other time period greater than one week.
- "Monthly Variation" or "Seasonal Variation" means within the year.

B.2.1.1 Seasonal and Monthly Variations

Seasonal fluctuations in traffic demand reflect the social and economic activity of the area being served by the highway. Figures B2-1 and B2-2 show monthly variation patterns observed in Illinois and Minnesota, and illustrate several significant characteristics.

- Monthly variations are more pronounced on rural routes than on urban routes.
- Monthly variations are more pronounced on rural routes serving primarily recreational traffic than on rural routes serving primarily business routes.
- Daily traffic patterns vary by month more for recreational routes.

These observations lead to the conclusion that commuter and business-oriented travel occurs in more uniform patterns, and that recreational traffic is subject to the greatest variation among trip-purpose components of the traffic stream.

The data for Figure B2-2 were collected on the same Interstate route. One segment is within 2 km of the central business district of a large metropolitan area. The other segment is within 80 km of the first, but serves a combination of recreational and intercity business travel. The wide difference in seasonal variation patterns for the two segments underscores the effect of trip purpose, and it may also reflect capacity restrictions on the urban section.

B.2.1.2 Daily Variation

Volume variations by day of the week are also related to the type of highway on which observations are made. Figure B2-3 shows that weekend volumes are lower than weekday volumes for highways serving predominantly business travel, such as urban freeways. In comparison, peak traffic occurs on weekends on main rural and recreational access facilities. Further, the magnitude of daily variation is highest for recreational access routes and least for urban commuter routes.

Figure B2-4 shows the variation in traffic by vehicle type for the right lane of an urban freeway. From this figure, it is evident that truck traffic is the most significantly reduced on weekends.

The extent of daily volume variation decreases as volume increases, often reflecting the effect of capacity restrictions on demand.

Although the values shown in Figures B2-3 and B2-4 are illustrative of typical patterns that may be observed, they are not meant to be a substitute for local studies and analysis.

B.2.1.3 Hourly Variation

Typical hourly variation patterns, as related to highway type and day of the week, are shown in Figure B2-5. The typical morning and evening peak hours are evident for urban commuter routes on weekdays. The evening peak is generally somewhat more intense than the morning peak. On weekends, urban routes show a peak that is less intense and more spread out, occurring in the early to mid-afternoon period.

Recreational routes also have single daily peaks. Saturday peaks on such routes tend to occur in the late morning or early afternoon (as travellers go to their recreational destination) and in late afternoon or early evening on Sundays (as they return home).

On inter-city routes serving a mix of traffic, late afternoon peaks are evident, and there is less difference between the variation patterns for weekdays and weekends.

The regularity of hourly variation is of great importance. The stability of peak-hour demands affects the viability of using such values in design and analysis of highways. Figure B2-6 shows data obtained over a 77-day period in metropolitan Toronto. The shaded area indicates the range within which one can expect 95% of the observations to fall. Although the variations by hour of the day are typical for urban areas, the relatively narrow and parallel fluctuations among the 77 days indicate the regularity of the basic pattern. The observations depicted were obtained from detectors measuring one-way traffic only, as evidenced by the single peak hour shown for either morning or afternoon.

It is noted that the data of Figures B2-5 and B2-6 are typical of observations that can be made. The patterns illustrated, however, do vary in response to local travel habits and environments, and these examples should not be used as a substitute for locally obtained data.

B.2.1.4 The Peak Hour

Capacity and other traffic analyses focus on the peak hour of traffic volume, because it represents the most critical period for operations and has the highest capacity requirements. The peak-hour volume, however, is not a constant value from day-to-day or from season-to-season.

If the highest hourly volumes for a given location were listed in descending order, a large variation in the data would be observed, depending on the type of route and facility under study.

Rural and recreational routes often show a wide variation in peak-hour volumes, with several extremely high volumes occurring on a few selected weekends or other peak periods, and with traffic during the rest of the year at much lower volumes, even during the peak hour. This occurs because the traffic stream consists of few daily or frequent users, with the major component of traffic generated by seasonal recreational activities and special events.

Urban routes, on the other hand, show very little variation in peak-hour traffic. The majority of users are daily commuters or frequent users, with occasional and special event traffic at a minimum. Further, many urban routes are filled to capacity during each peak hour, and variation is therefore severely constrained. In many urban areas, both the am and pm peak periods extend for more than one hour.

Figure B2-7 shows hourly volume relationships measured on a variety of highway types of Minnesota. Recreational facilities show the widest variation in peak-hour traffic, with values ranging from 30% of the Annual Average Daily Traffic (AADT) occurring in the highest hour of the year, to about 15.3% of AADT occurring in the 200th highest hour of the year and 8.3% in the 1,000th highest hour of the year. Main rural facilities also display a wide variation, with the highest hour subjected to 17.9% of the AADT, decreasing to 10% of the AADT in the 100th hour and 6.9% of the AADT in the 1,000th hour. Urban radial and circumferential facilities show far less variation, with the range in percent of AADT covering a narrow band, from approximately 11.5% for the highest hour to 7-8% for the 1,000th highest hour.
It should be noted that Figure B2-7 includes all hours, not just peak hours of each day.

B.2.2 Spatial Distribution

While traffic volume varies in time, it also varies in space. The two critical spatial characteristics of interest in capacity analysis are directional distribution and lane distribution. Volume may also vary longitudinally along various segments of a facility, but this does not explicitly affect capacity analysis. Each facility segment serving different traffic demands must be analyzed separately.

B.2.2.1 Directional Distribution

During any particular hour, traffic volume may be greater in one direction than in the other. An urban radial route, serving strong directional demands into the city in the morning and out of it at night, may display as much as a 2:1 imbalance in directional flows. Recreational and rural routes may also be subject to significant directional imbalances, which must be considered in the design process.

Directional distribution is an important factor in highway capacity analysis. Capacity and level of service on 2-lane rural highways vary substantially with directional distribution because of the interactive nature of directional flows on such facilities. Procedures for two-lane highways include explicit consideration of directional distribution.

It should also be noted that directional distribution is not a static characteristic in time. It changes by hour of the day, day of the week, season and from year-to-year. Development in the vicinity of highway facilities often induces traffic growth that changes the existing directional distribution.

B.2.2.2 Lane Distribution

When two or more lanes are available for traffic in a single direction, the distribution in lane-use will vary widely. The lane distribution will depend on traffic regulations, traffic composition, speed and volume, number and location of access points, origin-destination patterns of drivers, development environment, and local driver habits.

Because of these factors, there are no typical lane distributions. The procedures of this manual assume an average ideal capacity of multilane uninterrupted flow facilities of 2000 pc/h/l, recognizing that flow in some individual lanes will be higher and in others lower. Data collected indicate no consistency in lane distribution. For example, the peak lane on a six-lane freeway may be the right, centre or median lane, depending on local conditions. Heavier vehicles tend towards the right-hand lanes, partially because they operate at lower speeds than other vehicles, and because of regulations sometimes prohibiting them from using leftmost lanes.

Lane distribution is a critical factor in the analysis of freeway ramp terminals in as much as the traffic in the right lane forms the merge or diverge volume in conjunction with the ramp vehicles. Procedures for their analysis focus on estimating traffic in the right lane, as well as on truck presence in the lane.

B.2.3 Speed, Flow and Density

Speed, or its reciprocal travel time, is an important measure of the quality of traffic service provided to the driver. It is used as one of the more important measures of effectiveness defining levels of service for many types of facilities, such as rural two-lane highways, arterials, freeway weaving sections and others.

When used as a measure of effectiveness, speed criteria must recognize driver expectations and roadway function. Thus, a driver expects a higher speed on a freeway than on an urban arterial. Lower speeds will be tolerated on a roadway with more severe horizontal and vertical alignment, because drivers will not be comfortable driving at extremely high speeds. Level-of-service criteria are predicated on these and other influencing factors.

The relationship between speed and flow is illustrated in Figure B2-8.

B.2.3.1 Definitions

- Average running speed, also called "space mean speed", is a traffic stream measurement based on the observation of vehicle travel times travelling a section of highway of known length. It is defined as the length of the section divided by the average running time of vehicles to travel the section. "Running time" includes only time which vehicles spend in motion, and does not include stopped delays.

- Average travel speed is also a traffic stream measurement based on travel time observations over a known length of highway. It is defined as the length of the section divided by the average travel time of vehicles travelling the section, including all stopped delay times.

- Time mean speed is the arithmetic average of vehicle speeds observed passing a point on a highway, and it is also referred to as the "average spot speed." Individual speeds are recorded passing a point, and are arithmetically averaged.

- Average desired speed is the minimum speed that a driver would choose to drive at in a given environment.

- Free flow speed is the average speed of all vehicles over those portions of arterial segments that are not close to signalized intersections and are at low traffic volumes in which vehicles are not constrained by other vehicles. It is similar to average desired speed.

Most of the procedures using speed as a measure of effectiveness in this manual use "average travel speed" as the defining parameter. For uninterrupted flow facilities not operating at LOS F, (defined in B.3.2.1) the average travel speed is equal to the average running speed.

Volume and flow are measures that quantify the amount of traffic passing a point on a lane or roadway during a designated time interval. They are defined as follows:

- Volume is the total number of vehicles that passes over a given point or section of a lane or roadway during a given time interval, expressed in terms of annual, daily, hourly or subhourly periods.

- Flow is the equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval less than one hour, usually 15 min.

The distinction between volume and flow is an important one. Volume is the actual number of vehicles observed or predicted to be passing a point during a time interval. Flow represents the number of vehicles passing a point during a time interval less than one hour, but expressed as an equivalent hourly rate. Flow is found by taking the number of vehicles observed in a subhourly period and dividing it by the time (in hours) over which they were observed. Thus, a volume of 100 vehicles observed in a 15 min period gives a flow of 100 v/0.25 h or 400v/h.

- Density is the number of vehicles occupying a given length of a lane or roadway, averaged over time, usually expressed as vehicles per kilometre (v/km).

Direct measurement of density in the field is difficult, requiring a vantage point from which significant lengths of highway can be photographed, videotaped, or observed. It can be computed, however, from the average travel speed and flow, which are more easily measured.

$$v = S \times D$$

Where: v = flow, in v/h
S = average travel speed, in km/h and
D = density, in v/km.

B.2.3.2 Relationships

The relationship between speed and flow is illustrated in Figures B2-8 and B2-9.

As flow, speed and density are related by the expression $v = S \times D$, it follows that determination of a speed-density relationship also fixes the relationships between density and flow, and speed and flow.

If there is zero density, there can be no flow. If the roadway is at jam density (where speed is zero), there is likewise no flow.

As speed and flow are the most readily measured traffic stream parameters, and since speed has historically been a major measure of effectiveness in level-of-service analysis, the speed-flow curve is the most often calibrated from field data.

Figure B2-8 shows the results of a study of flow on four, six, and eight-lane freeways. The data were collected on New York area parkways under ideal conditions (no heavy vehicles, adequate geometrics). For both the four-lane and eight-lane parkways, there is little variation in speed up to flows of 1500 pc/h/lane, while data for the six-lane parkway are too sparse at flows below this level to draw firm conclusions.

A study of flow on a six-lane freeway near Toronto shows similar characteristics. As illustrated in Figure B2-9, speed is constant to a flow of about 1525 pc/h/lane. The figure illustrates, however, the difficulty in firmly fixing the shape and location of the curve beyond this range. Data points are scattered considerably, and any one of several curves (labelled A through E in Figure B2-9) could be fitted through the data, each with markedly differing characteristics. One theory suggests that perhaps speed is constant virtually to the point of capacity, after which breakdown occurs and unstable flow ensues. The data, collected at a bottleneck location, may suggest that some drivers forced to wait in a queue approaching the section simply do not accelerate to ambient speed, while those who do not have to wait in the queue travel the bottleneck at the free-flow speed, no matter what level of flow exists.

As a result of observations indicating little sensitivity of speed to flow over a substantial range of stable flows, density is used as the primary parameter defining multilane level of service. Time delay is used as the principal level-of-service parameter for two-lane highways.

B.2.4 Traffic Terms for Design

Standard terms for traffic characteristics are required to effect exchange of information and to quantify variations in traffic characteristics. The following definitions describe the terms used throughout this chapter.

B.2.4.1 Volume and Flow

Annual Average Daily Traffic (AADT)

$$\frac{\text{total traffic for entire year}}{365}$$

Monthly Average Daily Traffic (MADT)

$$\frac{\text{total traffic for one month}}{\text{number of days in that month}}$$

Summer Average Daily Traffic (SADT)

$$\frac{\text{total traffic during the summer period}}{\text{number of days in the summer period}}$$

Average Daily Traffic

$$\frac{\text{total traffic during any period} \geq \text{one day and} < \text{one year}}{\text{number of days in the period}}$$

Volume counts taken at any time of year can be adjusted to AADT by dividing the count by the factor "percent AADT" expressed as a decimal.

Short term intersection counts (typically 8-hour) can be adjusted to 24-hour counts if (a) traffic counter(s) is in operation at the same time by using the 24/8 counter ratio. The 24-hour count can then be adjusted to AADT.

<u>30th Highest Hour</u>

The 30th highest hourly volume occurring in one year, often used for design, and is often expressed as a percent of AADT.

<u>Directional Distribution</u>

The distribution of traffic volume or flow by direction, each direction expressed as a percent of the total. For 2-lane highways directional distribution is used to obtain an adjustment factor for level of service and capacity calculations. For the design hour volume calculation a factor of 1 is used.

For multi-lane highways, the higher value of the directional distribution is used to obtain an adjustment factor since analysis is done by direction.

Design Hour Volume is an hourly volume used for design. It may be expressed as a percentage of ADT or may be taken to be the 30th highest hour.

Peak Hour Factor (PHF) is used to take into account peaking within the design hour and is normally based on a 15-minute period.

$$\text{PHF} = \frac{\text{hourly volume}}{(\text{highest 15-minute volume}) \times 4}$$

Service Flow is the demand flow and is:

Service Flow = (highest 15-minute volume) x 4

It follows therefore that:

$$\text{Service Flow} = \frac{\text{Hourly Volume}}{\text{PHF}}$$

Maximum Service Flow is the maximum flow that a roadway can carry at a given level of service.

B.2.4.2 Vehicles and Terrain

The proportion of trucks, RV's and buses is required for use in applying the procedures in this chapter. Adjustments for each of these vehicle categories is given in terms of passenger car equivalents.

For general terrain segments adjustment factors are provided for three categories of terrain: level, rolling and mountainous. Where the prevailing terrain falls between two of these categories, interpolation of adjustment factors is appropriate.

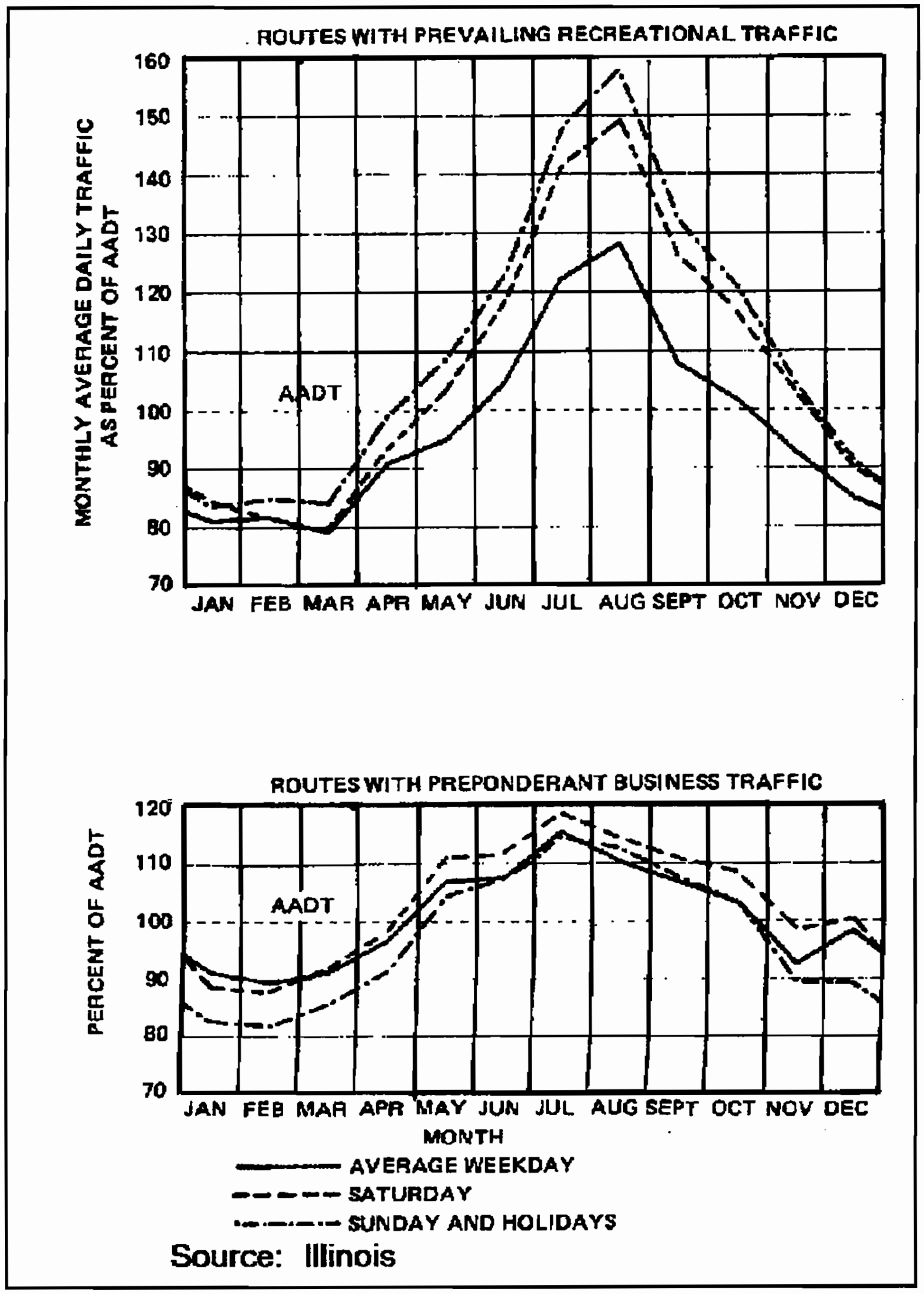


Figure B2-1

Variation of Average Daily Traffic By Month

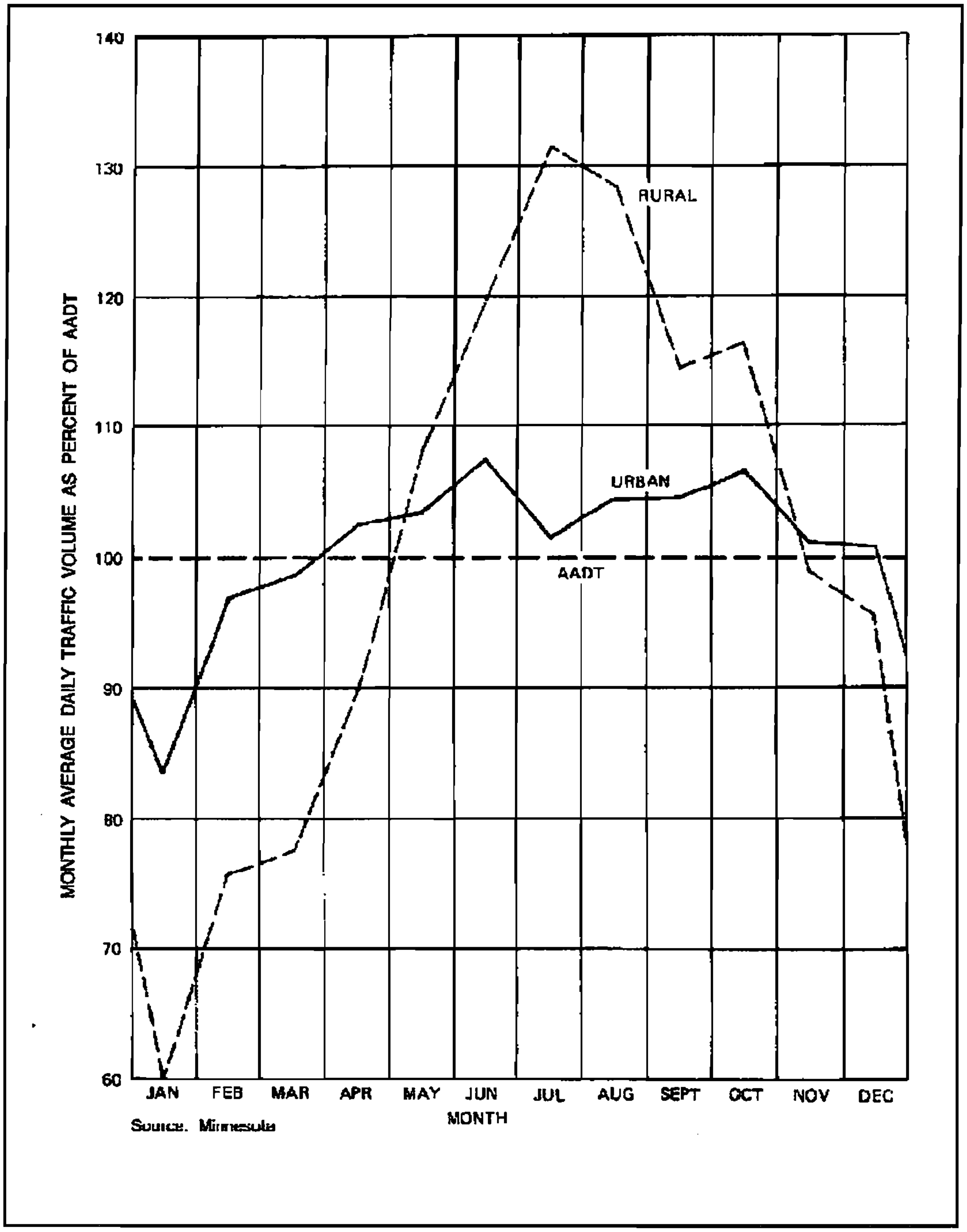


Figure B2-2

Variation of Average Daily Traffic By Month

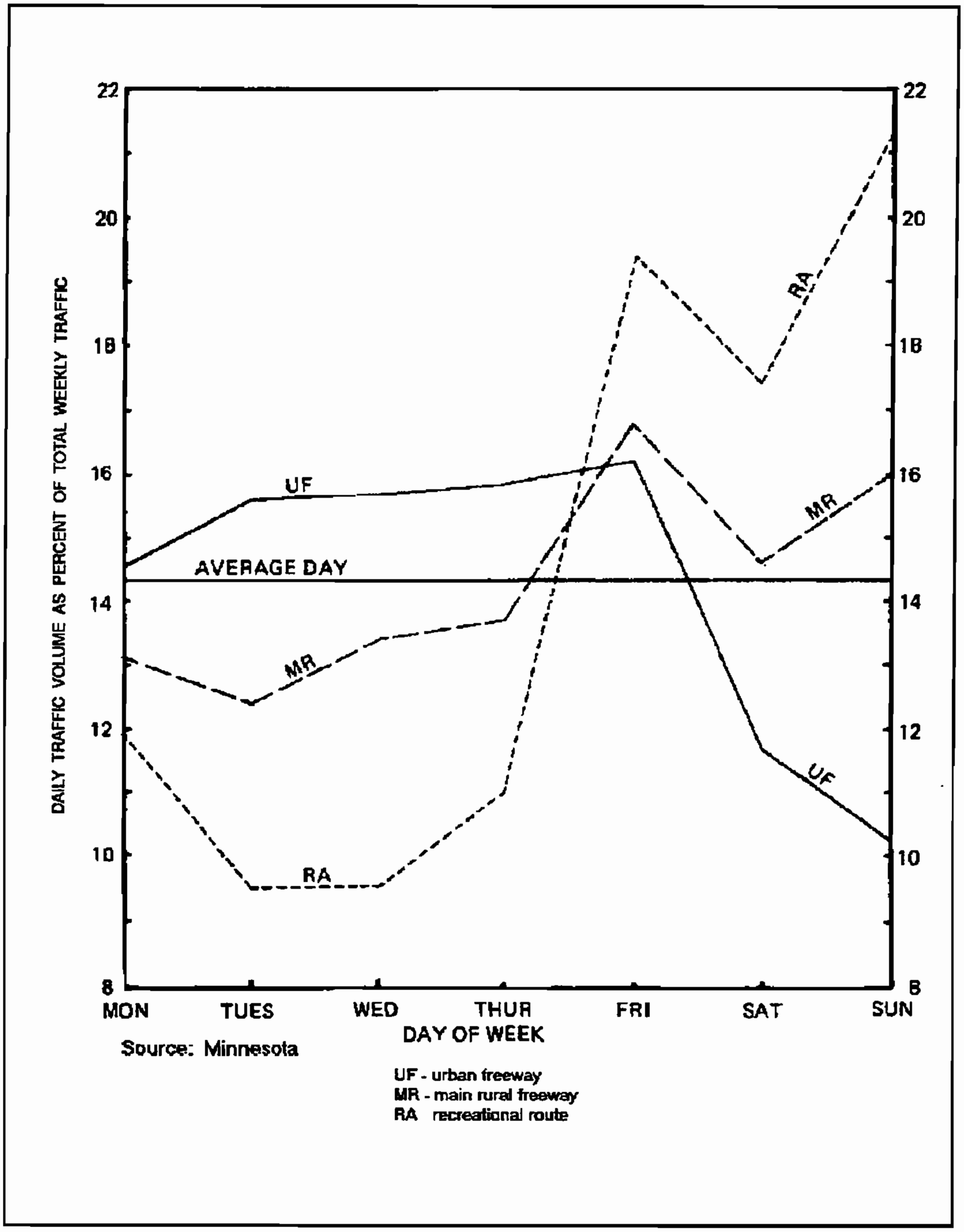
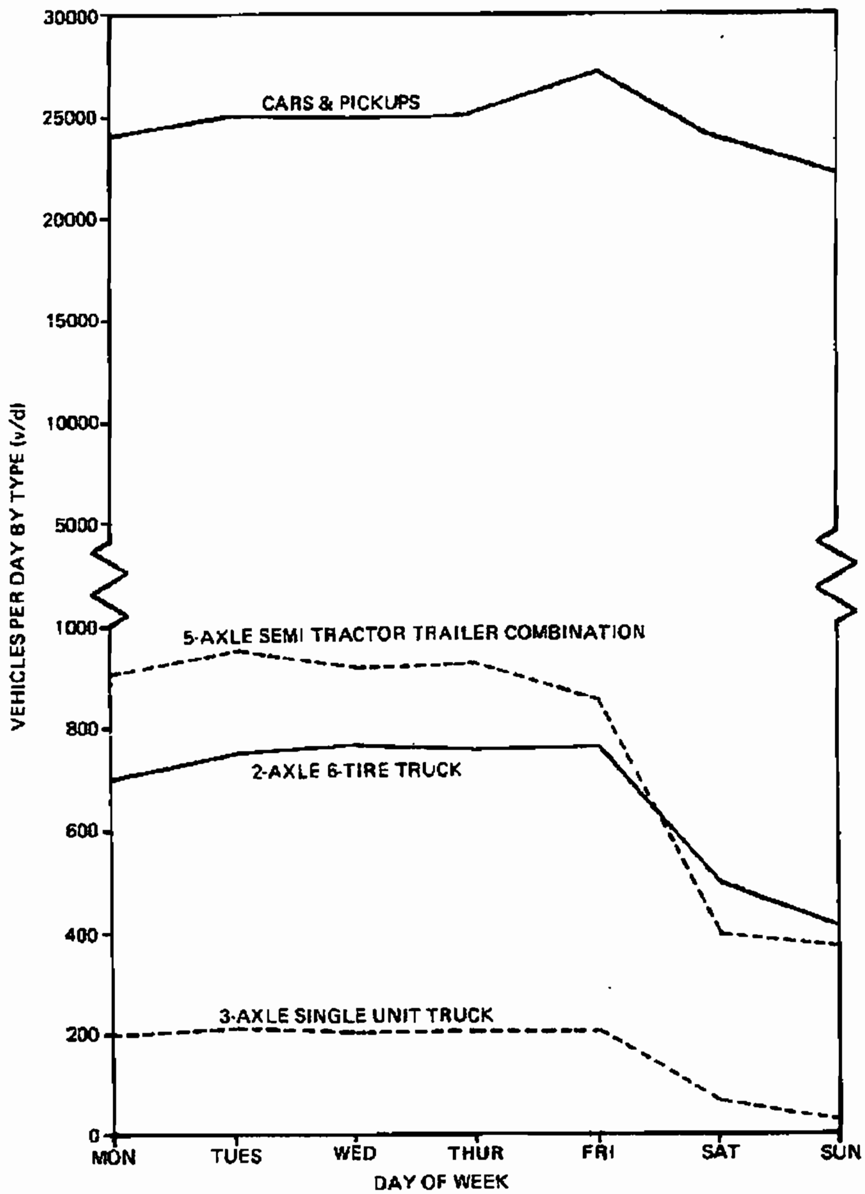


Figure B2-3

Daily Variation



Source: Minnesota

Figure B2-4

Daily Variation by Vehicle Type

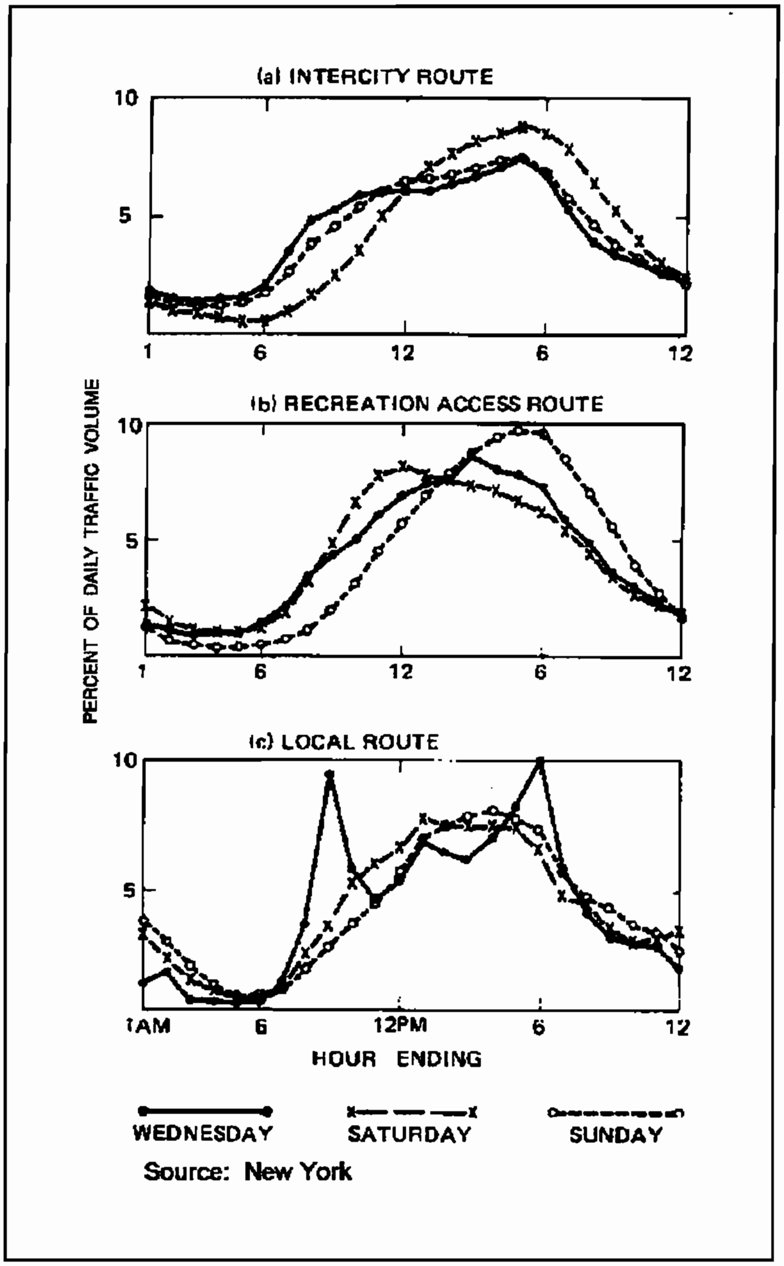
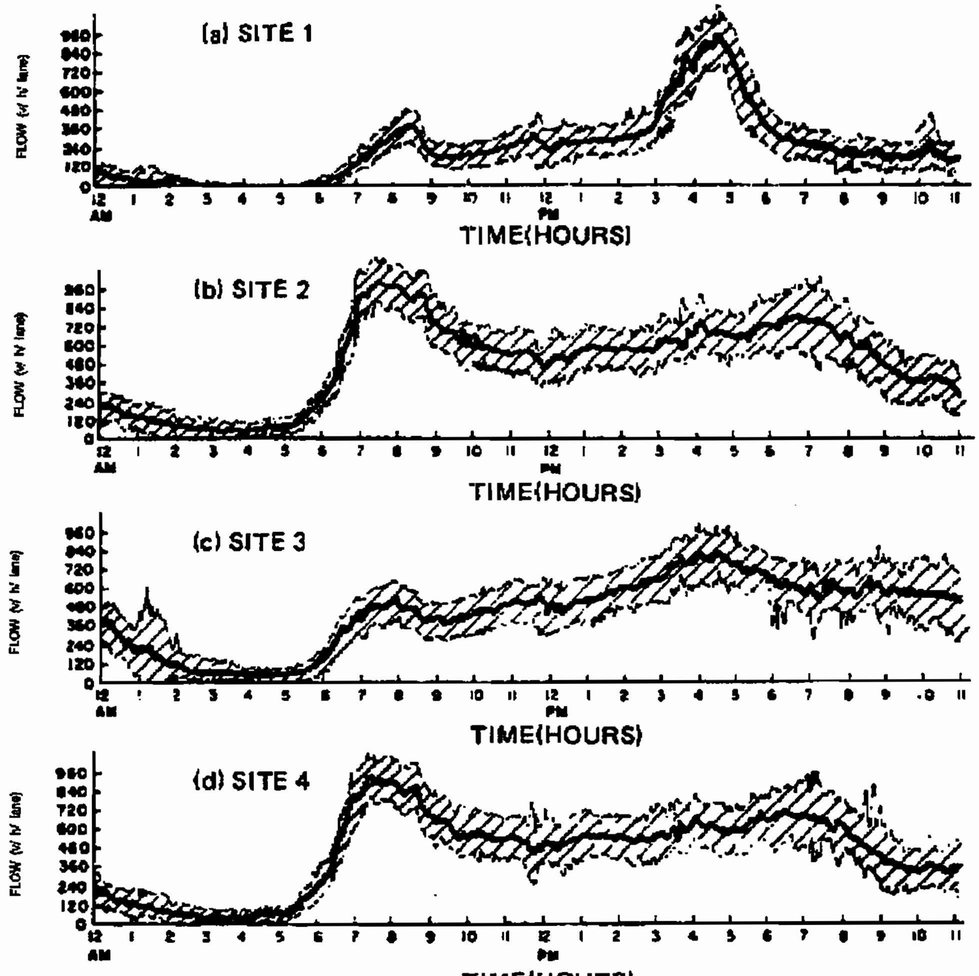


Figure B2-5

Hourly Variation on Rural Routes



Source: Toronto

NOTE: (1) SITES 2 and 4 ARE ONE BLOCK APART ON SAME STREET, IN SAME DIRECTION
(2) ALL SITES ARE TWO MOVING LANES

Figure B2-6

Hourly Variation on 2-Lane Urban Arterials

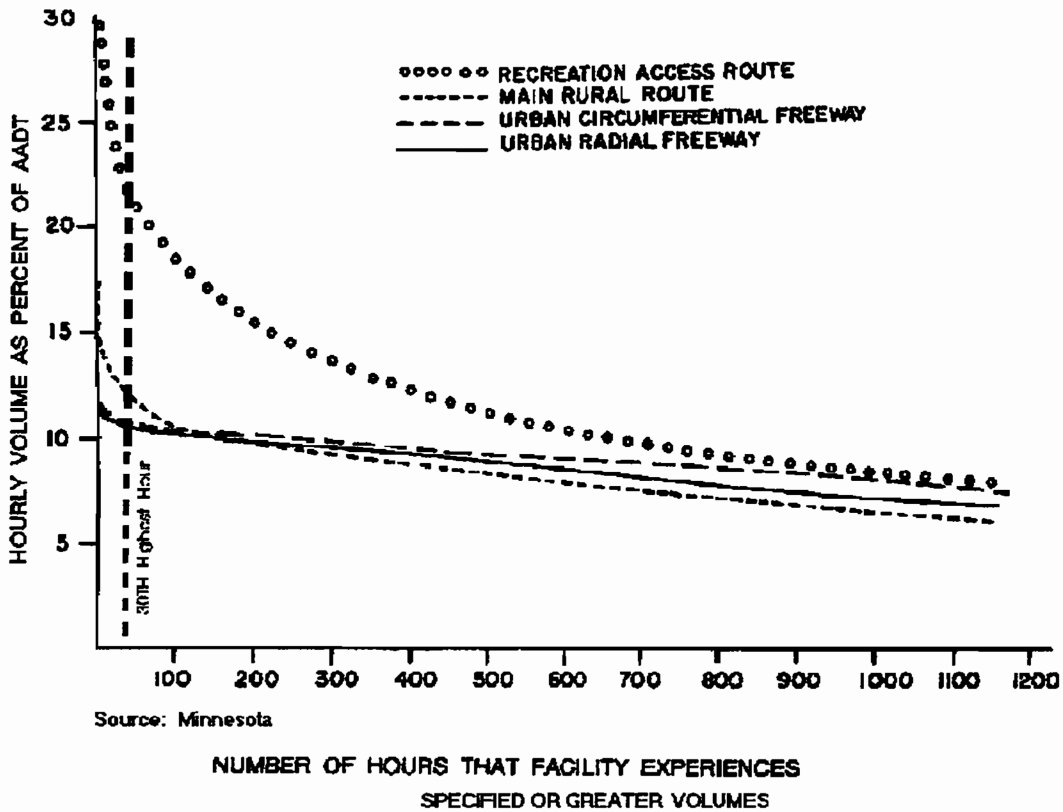


Figure B2-7

Ranked Hourly Volumes

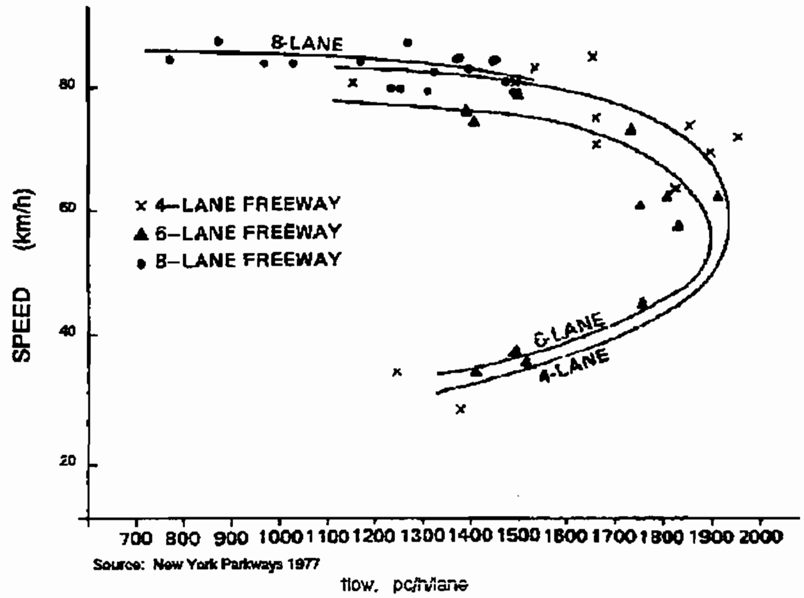


Figure B2-8

Relationship Between Speed and Flow

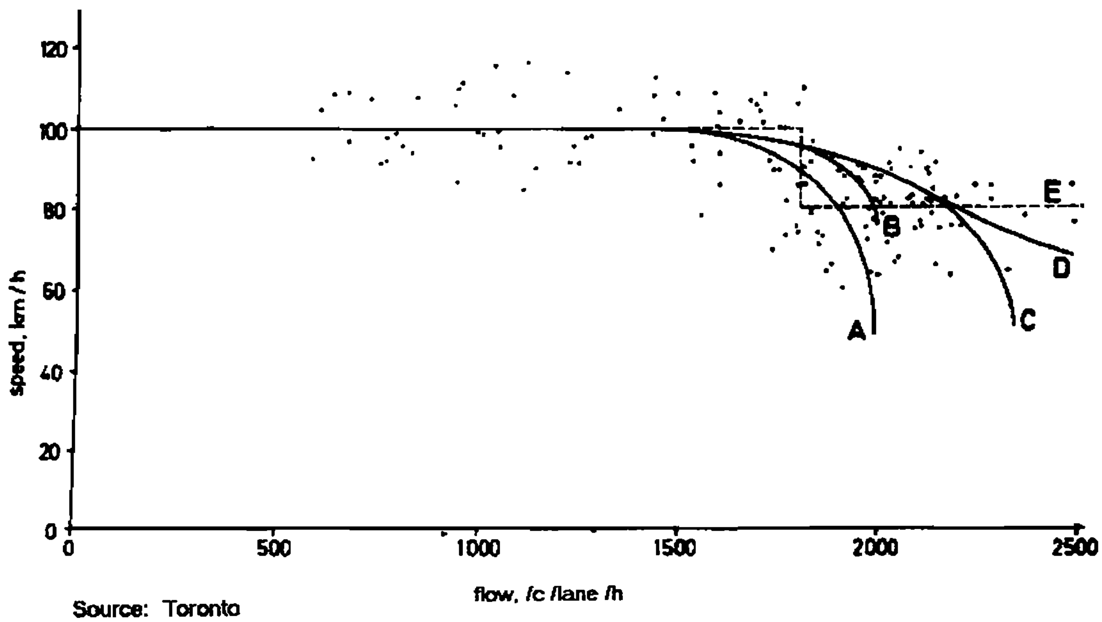


Figure B2-9

Relationship Between Speed and Flow

B.3 PRINCIPLES OF CAPACITY AND LEVEL OF SERVICE

A principal objective of capacity analysis is the estimation of the maximum amount of traffic that can be accommodated by a given facility. Capacity analysis would, however, be of limited utility if this were its only focus and traffic facilities generally operate poorly at or near capacity. Capacity analysis is also intended to estimate the maximum amount of traffic that can be accommodated by a facility while maintaining prescribed operational qualities.

The definition of operational criteria is accomplished using levels of service. Ranges of operating conditions are defined for each type of facility, and are related to amounts of traffic that can be accommodated at each level.

The following sections present and define the two principal concepts, capacity and level of service.

B.3.1 Capacity

In general, the capacity of a facility is defined as the maximum hourly flow at which persons or vehicles can reasonably be expected to pass a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions.

The time period used in most capacity analysis is 15 min, which is considered to be the shortest interval during which stable flow exists. Flow is expressed in vehicles/hour (v/h).

Capacity is defined for prevailing roadway, traffic and control conditions, which should be reasonably uniform for any section of facility analyzed. Any change in the prevailing conditions will result in a change in the capacity of the facility. The definition of capacity assumes that good weather and pavement conditions exist.

Roadway conditions refer to the geometric characteristics of the street or highway, including: the type of facility and its development environment, the number of lanes (by direction), lane and shoulder widths, lateral clearances, design speed, and horizontal and vertical alignments.

Traffic conditions refer to the characteristics of the traffic stream using the facility. This is defined by the distribution of vehicle types in the traffic stream, the amount and distribution of traffic in available lanes of a facility, and the directional distribution of traffic.

Control conditions refer to the types and specific design of control devices and traffic regulations present on a given facility. The location, type and timing of traffic signals are critical control conditions affecting capacity. Other important controls include "STOP" and "YIELD" signs, lane use restrictions, turn restrictions, and similar measures.

It is also important to note that capacity refers to a rate of vehicular or person flow during a specified period of interest, which is most often a peak 15-min period. This recognizes the potential for substantial variations in flow during an hour, and focuses analysis on intervals of maximum flow.

B.3.2 Level of Service

The concept of level of service is defined as a qualitative measure describing operational conditions within a traffic stream, and their perception by motorists and/or passengers. A level-of-service definition generally describes these conditions in terms of such factors as speed and travel time, freedom to manoeuvre, traffic interruptions, comfort and convenience, and safety.

Six levels of service are defined for each type of facility for which analysis procedures are available. They are given letter designations, from A to F, with level-of-service A representing the best operating conditions and level-of-service F the worst.

B.3.2.1 Definitions

The following definitions apply to uninterrupted flow facilities, including freeways:

> Level-of-service A represents free flow. Individual users are virtually unaffected by the presence of others in the traffic stream. Freedom to select desired speeds and to manoeuvre within the traffic stream is extremely high. The general level of comfort and convenience provided to the driver, passenger, or pedestrian is excellent.

Level-of-service B is in the range of stable flow, but the presence of other users in the traffic stream begins to be noticeable. Freedom to select desired speeds is relatively unaffected, but there is a slight decline in the freedom to manoeuvre within the traffic stream from LOS A. The level of comfort and convenience provided is somewhat less that LOS A, because the presence of others in the traffic stream begins to affect individual behaviour.

Level-of-service C is in the range of stable flow, but marks the beginning of the range of flow in which the operation of individual users becomes significantly affected by interactions with others in the traffic stream. The selection of speed is now affected by the presence of others, and manoeuvring within the traffic stream requires substantial vigilance on the part of the user. The general level of comfort and convenience declines noticeably at this level.

Level-of-service D represents high-density, but stable, flow. Speed and freedom to manoeuvre are severely restricted, and the driver or pedestrian experiences a generally poor level of comfort and convenience.
Small increases in traffic flow will generally cause operational problems at this level.

Level-of-service E represents operating conditions at or near the capacity level. All speeds are reduced to a low, but relatively uniform value. Freedom to manoeuvre within the traffic stream is extremely difficult, and it is generally accomplished by forcing a vehicle or pedestrian to "give way" to accommodate such manoeuvres. Comfort and convenience levels are extremely poor, and driver or pedestrian frustration is generally high. Operations at this level are usually unstable, because small increases in flow or minor turbulence within the traffic stream cause breakdowns.

Level-of-service F is used to define forced or breakdown flow. This condition exists wherever the amount of traffic approaching a point exceeds the amount which can traverse the point. Queues form behind such locations. Operations within the queue are characterized by stop-and-go waves, and they are extremely unstable. Vehicles may progress at reasonable speeds for several hundred metres or more, then be required to stop in a cyclic fashion. Level-of-service F is used to describe the operating conditions within the queue, as well as the point of the breakdown. It should be noted, however, that in many cases operating conditions of vehicles or pedestrians discharged from the queue may be quite good. Nevertheless, it is the point at which arrival flow exceeds discharge flow which causes the queue to form and level-of-service F is an appropriate designations for such points.

Typical conditions for the range of levels of service are illustrated in Figure B3-1.

B.3.2.2 Service Flows

The procedures of this chapter establish or predict the maximum flow which can be accommodated by various facilities at each level of service, except level-of-service F, for which flows are unstable. Thus, each facility has five service flows, one for each level of service (A through E), defined as follows:

The service flow is the maximum hourly rate at which persons or vehicles can reasonably be expected to pass a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions while maintaining a designated level of service. As to capacity, the service flow is generally taken for a 15-min time period.

Note that service flows are discrete values, while the levels of service represent a range of conditions. Because the service flows are defined as maximums for each level of service, they effectively define flow boundaries between the various levels of service.

B.3.2.3 Measures of Effectiveness

For each type of facility, levels of service are defined based on one or more operational parameters which best describe operating quality for the subject facility type. The parameters selected to define levels of service for each facility type are called "measures of effectiveness", and represent those available measures that best describe the quality of operation on the effectiveness used to define levels of service for each facility type.

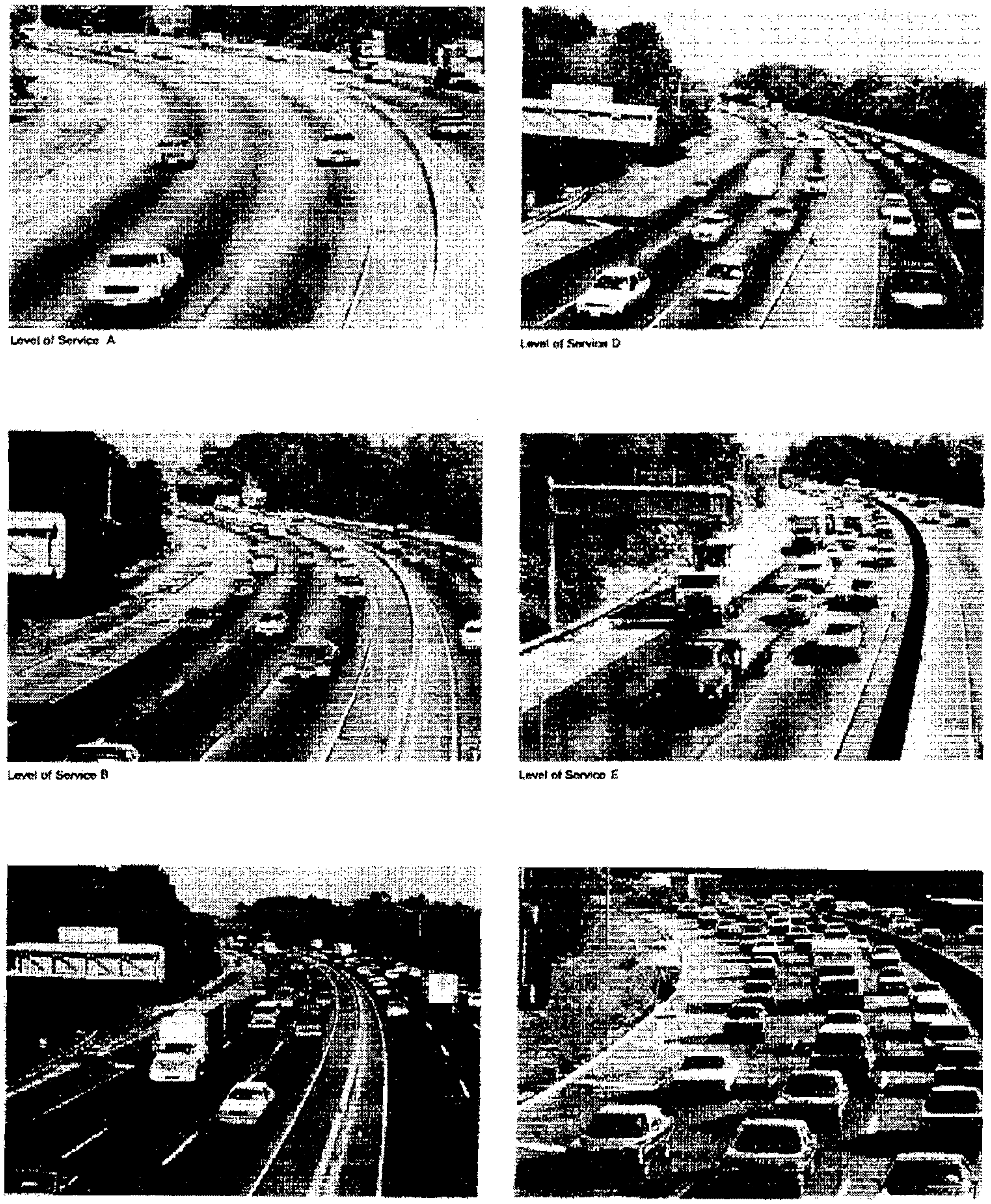
Each level of service represents a range of conditions, as defined by a range in the parameter(s) given in Table B3-1.

Thus, a level of service is not a discrete condition, but rather a range of conditions for which boundaries are established.

Table B3-1

MEASURES OF EFFECTIVENESS

Type of Facility	Measures of Effectiveness
Freeways:	
- Basic freeway segments	Density (pc/km/l lane)
- Weaving areas	Average travel speed (km/h)
- Ramp terminal	Flow (pc/h)
Multi-lane highways	Density (pc/km/l lane)
Two-lane highways	Delay (%)
Signalized intersections	Average individual stopped delay (s/v)
Unsignalized intersections	Reserve capacity (pc/h)
Urban and suburban arterials	Average travel speed (km/h)



Level of Service A

Level of Service B

Level of Service C

Level of Service D

Level of Service E

Level of Service F

Figure B3-1
Illustration of Levels of Service

B.4 RURAL HIGHWAYS

B.4.1 Uninterrupted Flow: Capacity and Level of Service

Uninterrupted flow facilities are defined as those without fixed interruptions such as traffic signals or other traffic control devices which are independent of traffic characteristics causing interruptions in the traffic flow. The term uninterrupted flow describes the type of facility rather than the characteristics of traffic flow which vary with traffic volume, climatic conditions, and geometric characteristics.

Rural highways are normally uninterrupted flow facilities, in which intersections are usually controlled by "stop" signs for the minor road and traffic signals, if any, are widely spaced. In general, a facility is considered to be one of uninterrupted flow if traffic signals or other fixed interruptions are more than 3 km apart.

For uninterrupted flow, capacity is defined as the maximum hourly rate at which persons or vehicles can be reasonably expected to pass a point on a lane or roadway during a given time period under prevailing roadway and traffic conditions:

The time period normally used for capacity analysis is 15 min and for the purpose of the definition, favourable weather and pavement conditions are assumed.

Roadway conditions refer to the geometric features of the highway, in particular classification, design speed, horizontal alignment, vertical alignment, cross section dimensions and the development environment of the area.

Traffic conditions refer to the distribution of vehicle types in the traffic stream and the distribution of traffic by direction.

For uninterrupted flow, level of service is defined as a qualitative measure of operational conditions by which drivers and passengers perceive their freedom to manoeuvre, comfort and convenience. These conditions are stated in terms of speed, density and traffic volume or flow. There are six levels of service designated A to F, A representing the best level of service and F the poorest. The levels of service are defined in Section B.3.2.1.

B.4.2 Two-Lane Highways

The parameters used to describe service quality of two-lane highways are:

- Time delay.
- Average travel speed.
- Utilization of capacity.

Time delay is the average time that all vehicles are delayed while travelling in platoons due to the inability to overtake, divided by the average total time. Time delay is expressed as a percentage. It is difficult to measure in the field and for practical purposes can be taken as the percentage of time that vehicles are travelling at headways of less than 5 s.

Average travel speed is the length of the section of highway under consideration divided by the average travel time of all vehicles travelling the entire segment in both directions during a designated time period.

Utilization of capacity is defined as the ratio of demand flow to capacity.

Level of service criteria refer to all three parameters with time delay being the primary measure and average travel speed and utilization of capacity being secondary.

The following procedures may be used to determine the level of service of an existing two-lane highway or future two-lane highway under given traffic and roadway conditions, and can also be used to determine what traffic volumes can be accommodated on two-lane highways for a given level-of-service and terrain conditions. The procedures are applicable to general terrain segments and to specific grades.

Figure B.4-1 shows the basic relationship between time delay, average speed and flow on two-lane highways for ideal conditions. It shows that speed is relatively insensitive to flow within the range of 0 to 2800 pc/h, while delay increases steadily as flow increases from 0% to 91% at capacity as passing demand exceeds passing capacity.

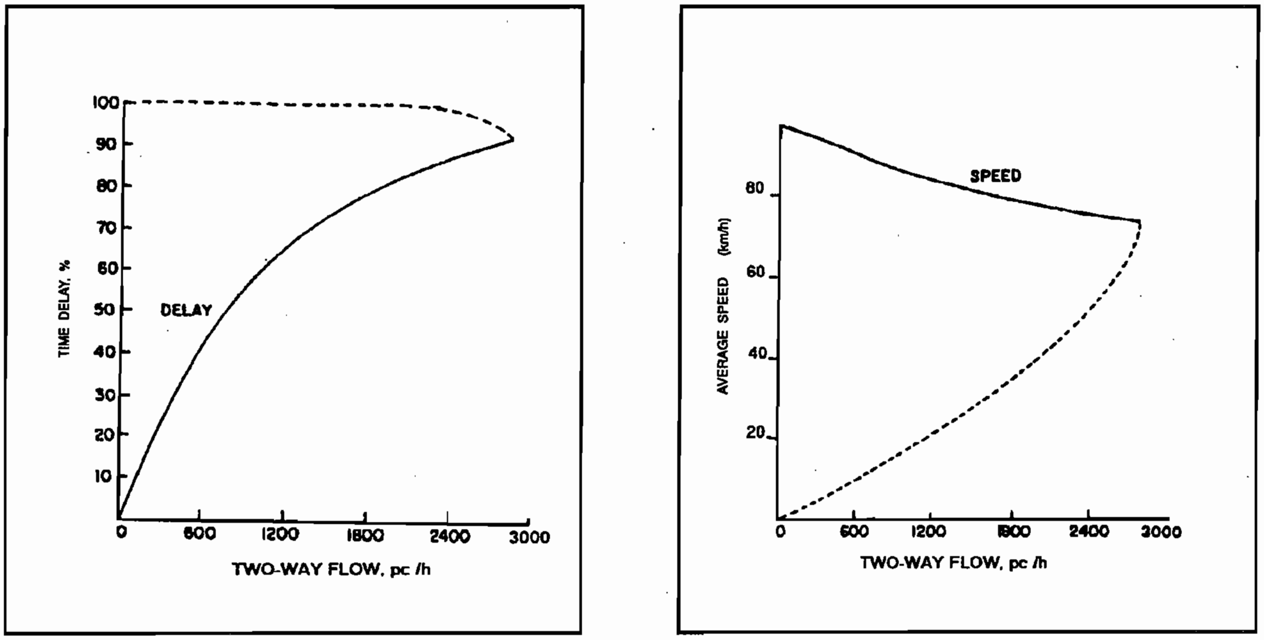


Figure B4-1

Relationship Between Time Delay, Average Speed and Flow on Two-Lane Highways

Ideal conditions for two-lane highways are defined in terms of geometric, traffic and environmental features. They are:

- Design speed equal to or greater than 100 km/h
- Lane width equal to or greater than 3.75 m
- Clear shoulders equal to or wider than 2.0 m
- No no passing zones
- All passenger cars in the traffic stream
- Directional distribution of 50/50
- No impediments to through traffic due to traffic control devices or turning traffic
- Level terrain

A no passing zone is defined as any marked no passing zone or any section of road wherein the passing sight distance, measured from the height of the driver's eye of 1.05 m to the pavement surface, is 450 m or less. No passing zone (NPZ) expressed as a percentage is the length of no passing zones divided by the total length of road multiplied by 100 and the average for both directions is used in the procedure.

The capacity of two-lane highways under ideal conditions is 2800 pc/h, total in both directions. This value, lower than the capacity of multi-lane uninterrupted flow highways of 2000 pc/h/lane is a reflection of the impact of opposing vehicles on passing opportunities and therefore on the ability to fill gaps in the traffic stream.

For analysis of two-lane highways, flows for the peak 15-min period within the hour are used. Thus where one-hour traffic counts or volumes are used they must be converted to 15-min flows using the peak hour factor (PHF), which is defined as the ratio of the one-hour volume (total for both directions) in vehicles per hour, to the flow for the peak 15-min period within the hour (total for both directions) in vehicles per hour.

$$v = \frac{V}{PHF}$$

where = v is flow
V is one-hour volume
PHF is Peak Hour Volume

The peak hour factor should be determined from traffic counts or other data where possible. Where such data are not available, the factors from Table B4-1 may be used. Where level of service is to be determined from volume, a value from part A of the table is selected. Where a service flow is to be determined, a value from part B of the table is used.

Table B4-1
PEAK HOUR FACTORS FOR TWO-LANE HIGHWAYS BASED ON RANDOM FLOW

A. LEVEL-OF-SERVICE DETERMINATIONS

Total 2-Way Hourly Volume v/h	Peak Hour Factor	Total 2-Way Hourly Volume v/h	Peak Hour Factor
100	0.83	1,000	0.93
200	0.87	1,100	0.94
300	0.90	1,200	0.94
400	0.91	1,300	0.94
500	0.91	1,400	0.94
600	0.92	1,500	0.95
700	0.92	1,600	0.95
800	0.93	1,700	0.95
900	0.93	1,800	0.95
		1,900	0.96

B. SERVICE FLOW DETERMINATIONS

Level of Service	A	B	C	D	E
Peak Hour Facot	0.91	0.92	0.94	0.95	1.00

The general expression for determining service flow for a given level of service for traffic operations on general terrain segments of two-lane highway is

$$SF_i = 2800 \times (v/c)_i \times f_d \times f_w \times f_{hv}$$

where:

SF_i = total service flow in both directions for prevailing roadway and traffic conditions, for level of service i in v/h;

$(v/c)_i$ = ratio of flow to ideal capacity for level of service i, obtained from Table B4-1;

f_d = adjustment factor for directional distribution of traffic, obtained from Table B4-3;

f_w = adjustment factor for narrow lanes and restricted shoulder width, obtained from Table B4-4;

f_{hv} = adjustment factor for the presence of heavy vehicles in the traffic stream, computed as:

$$^*f_{hv} = \frac{1}{1 + P_t(E_t - 1) + P_r(E_r - 1) + P_b(E_b - 1)}$$

where:

P_t = proportion of trucks in the traffic stream, expressed as a decimal;

P_r = proportion of RV's in the traffic stream, expressed as a decimal;

P_b = proportion of buses in the traffic stream, expressed as a decimal;

E_t = passenger-car equivalent for trucks, obtained from Table B4-5

E_r = passenger-car equivalent for RV's, obtained from Table B4-5

E_b = passenger-car equivalent for buses, obtained from Table B4-5

Level of service criteria are given in Table B4-2. For each level of service, the time delay is shown, defining the limits of each level of service. For information, the average speed is also shown, values varying with the nature of the terrain.

* If only one type of heavy vehicle is present, f_{hv} can be found from Table 3-9 in the Highway Capacity Manual, 1985.

Table B4-2

LEVEL-OF-SERVICE CRITERIA FOR GENERAL TWO-LANE HIGHWAY SEGMENTS

v/c RATIO[a]

LOS	% TIME DELAY	AVG SPD[b]	LEVEL TERRAIN						ROLLING TERRAIN							MOUNTAINOUS TERRAIN						
			NO PASSING ZONE %						AVG SPD	NO PASSING ZONE %						AVG SPD	NO PASSING ZONE %					
			0	20	40	60	80	100		0	20	40	60	80	100		0	20	40	60	80	100
A	30	93	0.15	0.12	0.09	0.07	0.05	0.04	92	0.15	0.10	0.07	0.05	0.04	0.03	90	0.14	0.09	0.07	0.04	0.02	0.01
B	45	88	0.27	0.24	0.21	0.19	0.17	0.16	87	0.26	0.23	0.19	0.17	0.15	0.13	87	0.25	0.20	0.16	0.13	0.12	0.10
C	60	84	0.43	0.39	0.36	0.34	0.33	0.32	82	0.42	0.39	0.35	0.32	0.30	0.28	79	0.39	0.33	0.28	0.23	0.20	0.16
D	75	80	0.64	0.62	0.60	0.59	0.58	0.57	79	0.62	0.57	0.52	0.48	0.46	0.43	72	0.58	0.50	0.45	0.40	0.37	0.33
E	75	72	1.00	1.00	1.00	1.00	1.00	1.00	64	0.97	0.94	0.92	0.91	0.90	0.90	56	0.91	0.87	0.84	0.82	0.80	0.78
F	100	72	-.-	-.-	-.-	-.-	-.-	-.-	64	-.-	-.-	-.-	-.-	-.-	-.-	56	-.-	-.-	-.-	-.-	-.-	-.-

a Ratio of flow rate to an ideal capacity of 2800 pc/h in both directions.
b These speeds are provided for information only and apply to roads with design speeds of 100 km/h and above.

Values for v/c in Table B4-2 are based on a directional distribution of 50/50. As the distribution varies further from the 50/50 "ideal" condition the two-way capacity is reduced. Values for adjustment, f_d are given in Table B4-3.

Table B4-3
ADJUSTMENT FACTORS FOR DIRECTIONAL DISTRIBUTION OF TWO-LANE ROADS

Directional Distribution	50/50	60/40	70/30	80/20	90/10	100/0
Adjustment Factor, f_d	1.00	0.94	0.89	0.83	0.75	0.71

Narrow lanes require drivers to drive closer to vehicles in the opposite lane. Similarly limited shoulder width and close roadside objects cause drivers to shy away from the right edge and drive closer to the opposing traffic stream. Both conditions cause drivers to reduce speed or increase headways, resulting in lower flows. Adjustment factors for the combined effect of narrow lanes and restricted shoulder width, f_w are given in Table B4-4.

Table B4-4

ADJUSTMENT FACTORS FOR THE EFFECT OF NARROW LANES AND RESTRICTED SHOULDER WIDTH, f_w

USABLE[a] SHOULDER WIDTH m	3.75-m LANES[b] LOS A-D	3.75-m LANES[b] LOS E	3.50-m LANES[b] LOS A-D	3.50-m LANES[b] LOS E	3.25-m LANES[b] LOS A-D	3.25-m LANES[b] LOS E	3.00-m LANES[b] LOS A-D	3.00-m LANES[b] LOS E	2.75-m LANES[b] LOS A-D	2.75-m LANES[b] LOS E
>2.0	1.00	1.00	0.97	0.98	0.92	0.93	0.83	0.85	0.72	0.77
1.5	0.97	0.98	0.92	0.96	0.86	0.91	0.78	0.84	0.69	0.76
1.0	0.96	0.96	0.84	0.93	0.79	0.89	0.72	0.82	0.62	0.74
0.5	0.80	0.93	0.76	1.89	0.71	0.84	0.63	0.79	0.66	0.70
0	0.71	0.81	0.67	0.84	0.62	0.81	0.56	0.74	0.49	0.66

a Where shoulder width is different on opposite sides of the roadway use the average shoulder width
b For analysis of specific grades use LOS E factors for all speeds less than 70 km/h

Values for v/c in Table B2-1 are based on a traffic stream consisting of passenger cars only. Vehicles having no more than four wheels are considered to be passenger cars and this includes vans and pick-up trucks. Heavy vehicles have the effect of reducing capacity and service flows, are divided into trucks, recreational vehicles and buses. Passenger-car equivalents for trucks (P_t), recreational vehicles (P_r) and buses (P_b) are given in Table B4-5. This Table is based on a 50/50 distribution of heavy (greater than 16000 kg) trucks and medium duty (less than 16000 kg) trucks. On two-lane highways where there is a predominance of heavy trucks, higher values of the passenger car equivalent should be used.

Table B4-5

AVERAGE PASSENGER-CAR EQUIVALENTS FOR HEAVY VEHICLES ON TWO-LANE HIGHWAYS

VEHICLE TYPE	LEVEL OF SERVICE	LEVEL	ROLLING	MOUNTAINOUS
Trucks, E_t	A	2.0	4.0	7.0
	B and C	2.2	5.0	10.0
	D and E	2.0	5.0	12.0
RV's E_r	A	2.2	3.2	5.0
	B and C	2.5	3.9	5.2
	D and E	1.6	3.3	5.2
Buses, E_b	A	1.8	3.0	5.7
	B and C	2.0	3.4	6.0
	D and E	1.6	2.9	6.5

B.4.3 Multi-Lane Highways

Multi-lane highways have two or more lanes in each direction and are not treated as freeways because they are undivided, do not have full control of access or both. They operate under uninterrupted flow conditions but not as efficiently as freeways because of various sources of side friction. Vehicles enter and leave the roadway at entrances and intersections, making right turns and left turns. Opposing vehicles on individual multi-lane roads are a further source of friction contributing to less efficient operation.

The procedures in the following paragraphs apply to rural divided and undivided multi-lane highways and to suburban highways in which traffic signal spacing is 3 km or more. Where signal spacing is less than 3 km, reference should be made to B.8 "Urban and Sub-Urban Arterials."

Level of service criteria for multi-lane highways are defined in terms of density. Density is a measure of the proximity of vehicles in the traffic stream and expresses the degree of manoeuvrability within the traffic stream.

Limiting values for density for each level of service are given in Table B4-6, stated in pc/km/lane.

The following procedures may be used to determine the level of service of an existing multi-lane highway under given traffic and roadway conditions, and can also be used to determine what traffic volumes can be accommodated on multi-lane highways for a given level of service and terrain conditions.

Figure B4-2 shows the basic relationship between density and flow for a range of design speeds. This represents a typical uninterrupted flow segment under ideal conditions and also reflects a maximum speed limit of 90 km/h.

Ideal conditions for multi-lane highways refer to geometric and environmental features and include:

- Lane width equal to or greater than 3.75 m.
- Clear shoulders equal to or wider than 2.0 m.
- No lateral obstruction within 2.0 m of the edge and travel lanes.
- All passenger cars.
- Divided highway cross section.
- Level terrain.

The general expression for determining service flow for a given level of service for traffic operations on general terrain segments of multi-lane highways is

$$SF_i = c \times N \times (v/c)_i \times f_w \times f_{hv} \times f_e \times f_p$$

where:

SF_i = service flow; the maximum flow that can be accommodated by the multi-lane highway segment in one direction, under prevailing roadway and traffic conditions, while meeting the performance criteria of LOS i, in v/h;

c = capacity for a multi-lane highway, pc/h/lane;

N = number of lanes in one direction;

$(v/c)_i$ = maximum volume-to-capacity ratio allowable while maintaining the performance characteristics of LOS i;

f_w = adjustment factor for lane width and/or lateral clearance restrictions;

f_{hv} = adjustment factor for the presence of heavy vehicles in the traffic stream, computed as:

$$f_{hv} = \frac{1}{1 + P_t(E_t - 1) + P_r(E_r - 1) + P_b(E_b - 1))}$$

if only one type of heavy vehicle is present, f_{hv} can be obtained from Table 3-9, HCM 1985.

f_e = adjustment factor for the development environment and type of multi-lane highway; and

f_p = adjustment factor for driver population.

where:

P_t = proportion of trucks in the traffic stream, expressed as a decimal;

P_r = proportion of RV's in the traffic stream, expressed as a decimal;

P_b = proportion of buses in the traffic stream, expressed as a decimal;

E_t = passenger-car equivalent for trucks, obtained from Table B4-8;

E_r = passenger-car equivalent for RV's, obtained from Table B4-8;

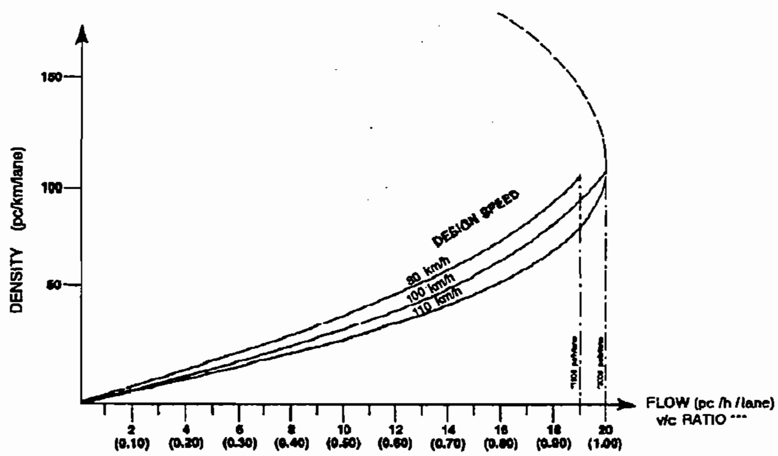
E_b = passenger-car equivalent for buses, obtained from Table B4-8.

The value for c, capacity is taken from Table B4-6. Values for v/c are taken from Table B4-7. The adjustment factor for the lane width and/or lateral clearance is found in Table B4-9. Lateral obstructions may be objects located at the roadside such as light standards, trees, signs, abutments, guide rails, bridge rails, and retaining walls. In Table B4-9 "obstruction on both sides of roadway" refers to one roadside and the median.

Table B4-6

CAPACITY OF MULTI-LANE HIGHWAYS

Design Speed km/h	Capacity, c pc/h
120	2000
110	2000
100	2000
90	1950
80	1900



* capacity
** relelects 80 km /h speed limit
*** v/c ratio based on capacity of 2000 pc /h /lane, applies only to 100 and 110 Km/h design speeds

Figure B4-2

Density-Flow Characteristics for Uninterrupted Flow Segments of Multilane Highways

Table B4-7

LEVEL OF SERVICE CRITERIA AND V/C RATIOS FOR MULTI-LANE HIGHWAYS

Level of Service	Density pc/km /lane	Design Speed km/h									
		120		110		100		90		80	
		Speed[a]		Speed		Speed		Speed		Speed	
		km/h	v/c	km/h	v/c	km/h	v/c	km/h	v/c	km/h	v/c
A	≤ 7	92	0.37	90	0.35	83	0.34	75	0.31		-.-
B	≤ 12	86	0.56	85	0.53	80	0.50	73	0.48	68	0.45
C	≤ 19	80	0.74	79	0.71	73	0.67	68	0.63	62	0.60
D	≤ 26	64	0.90	64	0.86	64	0.81	63	0.78	57	0.75
E	≤ 42	48	1.00	48	1.00	48	1.00	48	1.00	45	1.00
F	≤ 42		a		a		a		a		a

a = Average Travel Speed

Table B4-8

PASSENGER CAR EQUIVALENTS ON EXTENDED GENERAL MULTI-LANE HIGHWAY SEGMENTS

	TYPE OF TERRAIN		
FACTOR	LEVEL	ROLLING	MOUNTAINOUS
E_t (for trucks)	1.7	4.0	8.0
E_r (for RV's)	1.5	3.0	5.0
E_b (for buses)	1.6	3.0	4.0

Table B4-9

ADJUSTMENT FACTORS FOR RESTRICTED LANE WIDTH AND LATERAL CLEARANCE FOR MULTI-LANE HIGHWAYS

Distance from Edge of Travelled Way to Obstruction (m)[a]	ADJUSTMENT FACTOR									
	OBSTRUCTION ON ONE SIDE OF ROADWAY[b]					OBSTRUCTION ON BOTH SIDES OF ROADWAY[c]				
	LANE WIDTH (m)									
	3.75	**3.50**	**3.25**	**3.00**	**2.75**	**3.75**	**3.50**	**3.25**	**3.00**	**2.75**
4-LANE DIVIDED MULTI-LANE HIGHWAYS (2 LANES EACH DIRECTION)										
≥ 2.0	1.00	0.99	0.96	0.90	0.82	1.00	0.99	0.97	0.90	0.83
1.5	0.99	0.96	0.95	0.88	0.81	0.99	0.98	0.94	0.88	0.80
1.0	0.98	0.97	0.94	0.87	0.80	0.97	0.95	0.92	0.86	0.78
0.5	0.96	0.94	0.91	0.85	0.79	0.94	0.92	0.88	0.83	0.75
0.0	0.90	0.89	0.86	0.80	0.73	0.82	0.80	0.77	0.72	0.66
6-LANE DIVIDED MULTI-LANE HIGHWAYS (3 LANES EACH DIRECTION)										
≥ 2.0	1.00	0.99	0.95	0.88	0.79	1.00	0.99	0.95	0.88	0.79
1.5	0.99	0.97	0.93	0.86	0.77	0.99	0.97	0.93	0.86	0.77
1.0	0.98	0.96	0.92	0.85	0.76	0.97	0.95	0.91	0.84	0.76
0.5	0.97	0.95	0.91	0.84	0.75	0.95	0.93	0.89	0.82	0.74
0.0	0.94	0.92	0.89	0.83	0.74	0.91	0.90	0.86	0.79	0.70
4-LANE UNDIVIDED MULTI-LANE HIGHWAYS (2 LANES EACH DIRECTION)										
≥ 2.0	1.00	0.98	0.94	0.88	0.78	NA	NA	NA	NA	NA
1.5	0.99	0.97	0.93	0.86	0.77	NA	NA	NA	NA	NA
1.0	0.98	0.95	0.92	0.85	0.76	NA	NA	NA	NA	NA
0.5	0.94	0.92	0.89	0.82	0.73	0.93	0.90	0.87	0.83	0.78
0.0	0.88	0.86	0.84	0.78	0.70	0.81	0.80	0.77	0.72	0.66
6-LANES UNDIVIDED MULTI-LANE HIGHWAYS (3 LANES EACH DIRECTION)										
≥ 2.0	1.00	0.98	0.94	0.87	0.78	NA	NA	NA	NA	NA
1.5	0.99	0.97	0.92	0.86	0.77	NA	NA	NA	NA	NA
1.0	0.98	0.96	0.91	0.84	0.76	NA	NA	NA	NA	NA
0.5	0.96	0.94	0.89	0.83	0.74	0.96	0.93	0.89	0.82	0.75
0.0	0.94	0.92	0.87	0.80	0.72	0.92	0.90	0.85	0.78	0.70

a - Use the average distance to obstruction on "both sides" where the distance to obstructions on the left and right differs.

b - Factors for one-sided obstructions allow for the effect of opposing flow.

c - Two-sided obstructions include one roadside and one median obstruction. Median obstruction may exist in the median of a divided multilane highway or in the centre of an undivided highway which periodically divides to go around bridge abutments or other centre objects.

NA- No applicable; use factor for one-sided obstruction.

Adjustments for heavy vehicle, f_{hv}, consider three types: trucks, recreational vehicles (RV) and buses. The procedure is first to find the passenger car equivalent for each type of heavy vehicle using Table B4-9 and then to find the adjustment factor from the expression above. Where the ratio of trucks to the total number of RV's and buses exceeds 5:1, the RV's and buses may be treated as trucks.

Ideal conditions for multi-lane facilities refers to divided highways in rural development. The adjustment factor for development environment and type of highway, f_e accounts for reduction in service flows for undivided facilities and/or suburban development environment. The adjustment factor f_e may be found in Table B4-10.

Table B4-10

ADJUSTMENT FACTOR FOR TYPE OF MULTI-LANE HIGHWAY AND DEVELOPMENT ENVIRONMENT, f_e

TYPE	DIVIDED	UNDIVIDED
Rural	1.00	0.95
Suburban	0.90	0.80

Divided highways are those in which opposing flows are separated by a physical barrier or a flush median of at least 3 m in width.

The adjustment factor for suburban development depends on several factors including frequency of unsignalized intersections and development density.Judgement may be used to interpolate factors between rural and urban, and as a guide, more than 6 entrances per kilometre on one side of a highway can be regarded as suburban.

The adjustment factor for driver population; f_p, takes into account the familiarity of drivers with the road. Where the traffic stream contains a significant proportion of unfamiliar drivers, such as recreational drivers, the service flows are reduced and Table B4-11 gives values which may be applied based on local knowledge of driver population.

Interpolation between the factors may be used where conditions are not clearly defined.

Table B4-11

ADJUSTMENT FACTOR FOR DRIVER POPULATION

DRIVER POPULATION	FACTOR f_p
Commuter, or Other Regular Users	1.00
Recreational, or Other Nonregular Users	0.75 - 0.90

B.4.4 AUXILIARY LANES

Auxiliary lanes provide cost effective solutions to the common operational problem of slow moving vehicles on our roadways. Two of these, climbing and passing lanes, although similar in general appearance, have very different design procedures.

For Passing/Climbing Lane automated software, see Section C.4.2.4 of Chapter C.

Terms for Operational Analysis & Design

The following abbreviations describe the terms used throughout this section:

APO - Assured Passing Opportunity, expressed as percentage; it is a product of headway factor (HF) and the passing opportunity sight distance (POSD).

HF - Headway Factor, the percentage of total time when headway (H) equals or exceeds 25s in the opposing direction.

H - Headway, in seconds, the time interval between successive vehicles measured from leading edge to leading edge as they pass a given point.

LF - Lane Frequency (spacing of passing lanes)

NPZ - No passing Zone, any section of road wherein the passing opportunity sight distance (POSD) is 450 m or less. NPZ expressed as a percentage is the length of no passing zones divided by the total length of road multiplied by 100 and the average for both directions is used in the procedure.

PLL - Passing lane length.

PML - Passing Manoeuvre Length, the actual length to allow (Q) vehicles to perform a passing manoeuvre. This length includes a safety factor of 25%.

PO - Passing Opportunity, a function of sight distance.

POSD - Passing Opportunity Sight Distance, expressed as percentage; sight distance of 450 m or greater to enable one vehicle to pass another vehicle safely without restriction, measured from the driver's eye height of 1.05 m to the pavement surface. This percentage value can be determined graphically from plans and profiles.

Q - Number of vehicles in queue or platoon.

QFD - Queue Formation Distance, a distance required for (Q) vehicles to form a queue or platoon behind a slow moving vehicle.

S - Operating speed. (The passing lane criteria assumes an operating speed of 120 km/h.)

ΔS - Speed differential between the operating speed and the speed of slow moving vehicles. (The passing lane criteria assumes a speed differential of 30 km/h.)

TCL - Truck Climbing Lane

Vadv - Advancing traffic volume, in vehicles per hour (v/h). The passing lane criteria assumes Vadv = ½ DHV = Vopp.)

Vopp - Opposing traffic volume, in vehicles per hour (v/h.)

B.4.4.1 Climbing Lanes

Slow moving vehicles, in particular heavy trucks and recreational vehicles, can impede traffic flow and pose a safety hazard on significant upgrades. In these cases, the recommended safety improvement is a climbing lane. This is an extra lane dedicated specifically to slow moving vehicles. Slow moving vehicles are directed to travel in the right lane, allowing other drivers to pass on the left in the through lane.

The final decision on the climbing lane provision and its length should be based on a cost-effective investigation. This can be carried out using the computer program for the estimation of cost effectiveness of climbing lanes.

B.4.4.1.1 Warrants on Two-Lane Highways

The warrant for a truck climbing lane is based upon the speed reduction or level of service drop experienced on the upgrade. A climbing lane is warranted if each of the following criteria is satisfied:

1. One of the following conditions exists:
   - level of service E or F exists on the grade.
   - a reduction of two or more levels of service is experienced when moving from the approach segment to the grade.
   - a 15 km/h or greater speed reduction is expected for a typical heavy truck.
2. Upgrade traffic flow exceeds 200 v/h.
3. Upgrade truck flow exceeds 20 v/h.

All of these criteria, except for the 15 km/h speed reduction, are based upon the drivers' perception of how well the highway is performing. The speed reduction of 15 km/h is based upon safety. It has been shown that above a 15 km/h speed differential, accident rates increase significantly. For this reason, speed reduction is critical, thus it should always be foremost in the designer's mind.

The upgrade traffic flow is determined by multiplying the predicted or existing design hour volume by the directional distribution factor for the upgrade direction and dividing the result by the peak hour factor (the peak hour and directional distribution factors are discussed in Section B.2.4.1.. The number of upgrade trucks is obtained by multiplying the upgrade traffic flow by the percentage of trucks in the upgrade direction.

Most of the data required for assessing the criteria can be obtained from traffic counts. The speed reduction is estimated using a vehicle performance on grade graph (speed-distance curves), for the design vehicle. When using these graphs, the lane should begin at the point at which the speed drops to 15 km/h below the operating speed (critical length of grade) and continues until the speed increases to within 15 km/h of the operating speed.

Vehicle performance on grade graphs and vehicle specification are available in the Appendix to Chapter J.

The performance graphs are for the "typical" entry speed of 90 km/h and are for 60, 120, 180 and 210 kg/kW vehicles. If these do not well represent the situation under consideration, graphs for different entry speeds and different power to weight ratios are included in report TDS-90-12, available from the Research and Development Branch.

The minimum length of the climbing lane is 1500 m including tapers that are to be provided at the entrance and exit of the climbing lane.

B.4.4.1.2 Warrants on Multi-lane (4 Lanes or more) Highways

On a multi-lane highway a drop in the level of service or a speed reduction of 15 km/h (critical length of grade), on the upgrade, should be regarded as an indication that a climbing lane may be required. To verify this, an operational analysis of the roadway should be performed, both for the approach segment and the upgrade.

Should this analysis reveal that an additional lane is required on the upgrade, this is an indication that a climbing lane is warranted. However, the designer should then determine what the expected level of service is, on the upgrade, in the mixed vehicle lanes, after the climbing lane is constructed, to ensure that this will bring them up to the desired level of service.

To determine the capacity of the climbing lane and the number of trucks which can realistically be expected to use it, use the following equations:

$$c_T = 2000/E_T$$

where

c_T is the capacity of the climbing lane in v/h

and

E_T is the passenger car equivalent of trucks (Table B4-9).

The climbing lane should operate at approximately the same v/c ratio (i.e. level of service) as the rest of the roadway, thus select the v/c ratio corresponding to the desired design level of service. The service flow for the climbing lane can then be calculated:

$$SF_T = c_T(v/c)_i$$

where

SF_T is the service flow for the truck lane

and $(v/c)_i$ is the v/c for LOS_i

This is the number of trucks that can reasonably be assumed to use the climbing lane. The remaining heavy vehicles will share the mixed traffic lanes.

An example of this procedure is presented in sample calculations at the end of the section.

B.4.4.2 Passing Lanes

The inability to execute a passing manoeuvre on two-lane rural highways is a frequent cause of driver frustration. Passing opportunities are influenced mainly by sight distance, however, traffic in the opposing direction often determines whether or not a pass can be successfully completed. Frustration due to lack of passing opportunities is particularly common when the highway facility has some or all of the following characteristics:

- rolling terrain
- sparse development
- a high percentage of long-distance, high-speed trips
- a significant percentage of slow-moving vehicles generating platoons
- traffic volumes high enough to restrict passing but too low to warrant widening to four lanes.

These conditions are typical of many highways in northern Ontario.

Conventional passing opportunity (PO) is a function of sight distance alone. Assured passing opportunity (APO) modifies passing opportunity by a factor which accounts for the effect of opposing traffic in preventing passes despite adequate sight distance. Assured passing opportunity is always less than passing opportunity and reduces as traffic volumes increase.

An assured passing opportunity is defined as a condition where one vehicle may safely pass another without restriction either by visibility or opposing traffic.

The available micro-computer program (PCL) enables an expedient and comprehensive assessment of passing and climbing lane projects, analysis of potential sites as passing or climbing lane and priorities to be established between sites based on economic feasibility.

B.4.4.2.1 Warrants

The warrant for a passing lane is based on the available assured passing opportunity (APO) and an acceptable platoon length (Q). This length is normally taken to be 6 vehicles based on observation, including the slow moving vehicle at the head of the platoon, but could range from 4 to 8 vehicles.

The available APO for operation without passing lanes is computed with the projected required APO with passing lanes, based on the design year volume.

The available APO percentage is given by:

$$APO = (100\text{-}NPZ)HF$$

No passing zone is a function of the geometry of the road and in most cases it is a section of road where the passing sight distance is 450 m or less, measured from height of eye of 1.05 m to the height of object of zero.

Some situations, however, require a longer passing zone. In the northern regions often there are convoys of logging trucks travelling together. In these regions then, on highways which have a significant number of these convoys, an increased passing opportunity sight distance (POSD) of 1000m should be used.

Headway factor (HF) is the percentage of time where headway between opposing successive vehicles exceeds 25s. Headway factors may be obtained from Figure B4-3, which represents headway factors of 25s and 50s. As with the passing opportunity sight distance the headway factor needs to be adjusted to accommodate the convoys. A headway of 50s should be used in this case.

The required APO percentage is calculated using:

$$APO = \frac{PLL}{QFD + PLL}$$

The required APO may be obtained directly from Figure B4-4.

PLL the recommended maximum and minimum passing lane lengths are shown in Figure B4-5.

QFD is the queue formation distance required for Q vehicles to form a platoon behind a slow moving vehicle and can be calculated:

$$QFD = \frac{QS^2}{V_{adv}(\Delta S)}$$

where Q is the acceptable queue length
V_{adv} is the advancing traffic volume
S is the operating speed in km/h
ΔS is the speed differential in km/h.

This equation may be simplified to:

$$QFD = \frac{480Q}{V_{adv}}$$

The simplified equation assumes an operating speed of 120km/h and a differential of 30km/h. This should be satisfactory for most situations, but designers may wish to use the general equation should they feel their situation is not well represented by average values.

After it has been decided that passing lanes are warranted, the APO provided with the new lane may be calculated using:

$$APO_{WA} = \frac{P_1W_1 + P_2W_2 + \ldots P_nW_n}{W_1 + W_2 + \ldots W_n}$$

where

APO_{WA} is the weighted average of APO,
P_n is the APO available and
W_n is the length of the section.

B.4.4.2.2 Design Standards

The passing lane length is generally the length required to allow a platoon of Q vehicles to overtake a slow moving vehicle. As a general guide, a length in the range of 1500 m to 2000 m is recommended, see Figure B4-5. The recommended minimum length of 1500 m has been based on the required passing manoeuvre length (PML) and the average speed of passing vehicles.

The interval between successive passing lanes is influenced by driver frustration and economic constraints. To minimize frustration, the interval should be as short as possible. This frustration can be minimized by advance signs indicating the location of the next passing lane. Lane frequency (LF) (i.e. distance between passing lanes) can be calculated using:

$$LF = QFD + PLL$$

Lane width for passing lanes and width of adjacent shoulders are given in Chapter D, Section D.2.3 and D.5.3.

B.4.4.2.3 Lane Obsolescence

If the advancing traffic volume and the percentage of slow moving vehicles is high, the introduction of a passing lane provides less benefit to the level of service. This is a condition referred to as lane obsolescence. In this case, improvement to the level of service can only by achieved by widening to 4 lanes.

Lane obsolescence can be checked by use of the expression:

$$V_l = 600 - 10.6s$$

where

V_l is the advancing traffic volume, in v/h without lane obsolescence and

S is the percentage of slow-moving vehicles, in %.

If the prevailing advancing volume exceeds the volume, V_l , determined from this expression, a condition of lane obsolescence is reached.

If the prevailing advancing volume is less than the lane obsolescence condition, the level of service is satisfactory.

Calculated examples of the warrant procedure are presented at the end of this section.

All equations presented here are from Chapter 13 of the "Highway Capacity Manual Seminar Notes, 1985." If the designer wishes more detail, this document should be referred to.

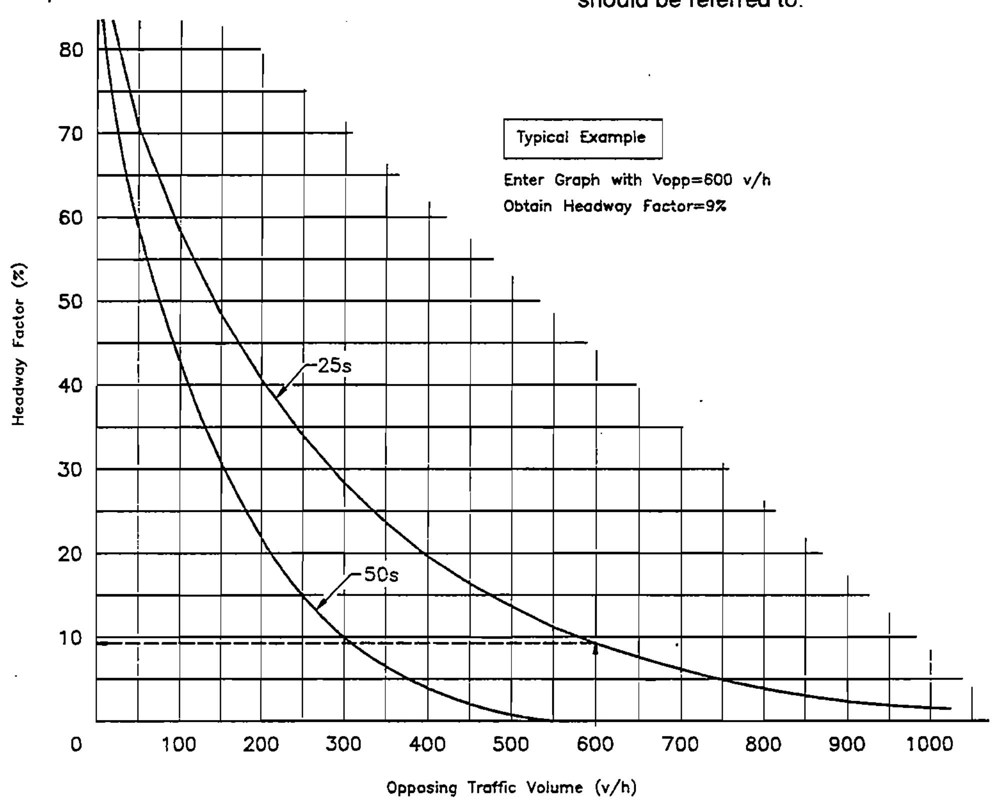


Figure B4-3
Headway Factors versus Opposing Traffic Volumes

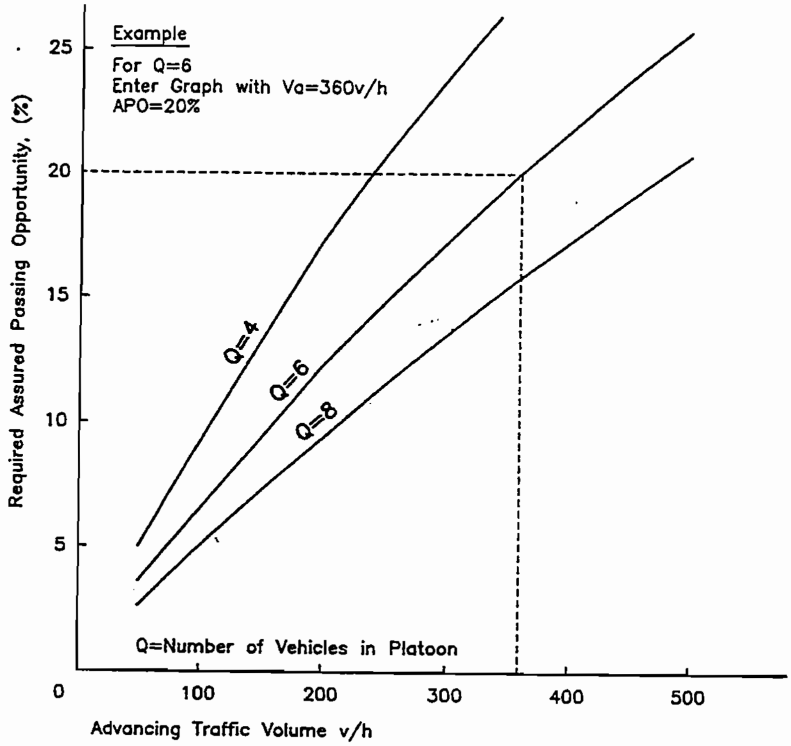


Figure B4-4
Required Assured Passing Opportunity versus Advancing Traffic Volumes

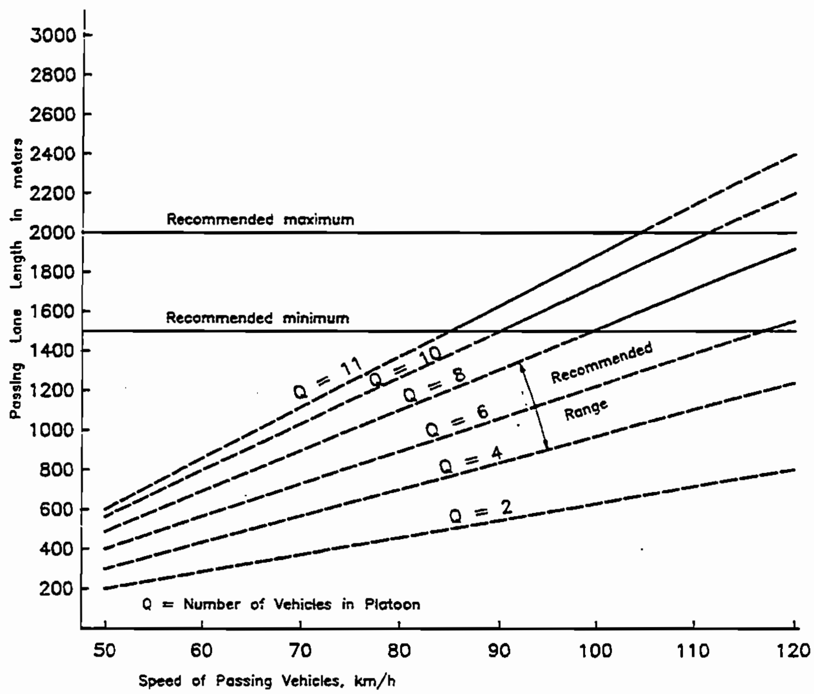


Figure B4-5
Passing Lane Length versus Speed of Passing Vehicles, for Various Lengths of Platoon

B.4.4.3 Calculated Examples

The following three examples demonstrate the operational analysis of a truck climbing lane on a multi-lane highway, a passing lane on a two-lane highway and a section of two-lane highway with existing climbing lanes.

No. 1 Operational Analysis of a Truck Climbing Lane on a Multi-lane Highway

Given:
- DHV of 2100v/h (weekday traffic)
- 20% trucks
- PHF 0.95
- 8 km of level terrain followed by 1.5km of 3% grade

Required: How many lanes must be provided to maintain LOS B?

Solution: Assume 120km/h design speed, 3.75m lanes and no lateral obstructions.

SF= DHV / PHF
=2100 / 0.95 = 2211v/h

c = 2000pc/h (per lane) (Table B4-6)

v/c = 0.56 (Table B4-7)

f_w = 1.00 (Table B4-8)

f_p = 1.00 (Table B4-11)

E_T(level) = 1.7 (Table B4-9)

E_T(upgrade) = 6 (Interpolated)

$$f_{HV}\ (level) = \frac{1}{1+P_T(E_T-1)}$$

$$= \frac{1}{1+0.2(1.7-1)} = 0.88$$

$$f_{HV}\ (upgrade) = \frac{1}{1+0.2(6-1)} = 0.50$$

Using the general expression for determining service flow (SF), compute the number of lanes in one direction.

Then:

$$N(\text{level}) = \frac{SF}{c(v/c)f_w f_{HV} f_p}$$

$$= \frac{2211}{2000(.56)(1)(.88)(1)} = 2.2\ lanes$$

N(upgrade) =

$$= \frac{2211}{2000(.56)(1)(.5)(1)} = 3.9\ lanes$$

These results suggest that the design should consist of a six lane facility, with a potential climbing lane on the upgrade. This should be checked using the special procedure for climbing lanes, as follows:

c_T = 2000/E_T
= 2000/6 = 333 trucks/hour

Using the design v/c value, it could be expected that the following number of trucks would use the lane:

SF_T = c_T(v/c)
= 333(0.56) = 186 trucks/hour

Thus the remaining freeway lanes would serve 2100 - 186 = 1914v/h, of which (2100x0.2) - 186 = 234v/h are trucks (12.2%). A computation for the design of the remaining lanes must therefore be conducted for a DHV of 1914 and 12% trucks.

SF = 1914/0.95 = 2015v/h

E_T = 5

$$f_{HV} = \frac{1}{1+0.12(5-1)} = 0.68$$

$$N = \frac{2015}{2000\ (.56)\ (1)\ (.88)\ (1)} = 2.6\ lanes$$

As the requirement for the remaining mixed vehicles in traffic lanes on the upgrade is less than three lanes, the design of a six-lane facility with a climbing lane is appropriate.

No. 2 Operational Analysis of a Passing Lane

Given:
- two-lane two way highway
- DHV 720 v/h
- no passing zone 30% (NPZ)
- slow moving vehicles 12%
- directional distribution 50/50 split

Required: are passing lanes warranted and if so, at what interval?

Solution: Opposing traffic volume = 360 v/h

HF = 23% (Fig. B4-3)

$$\text{Available APO} = (100\text{-NPZ})HF = (100-30)(0.23) = 16.1\%$$

$$\textit{Required APO} = \frac{PLL}{QFD + PLL}$$

(or Fig. B4-4)

Select a passing lane length of 2 km
Advancing traffic volume = 360 v/h

$$QFD = \frac{480(2)}{V_{adv}} = \frac{480(6)}{360} = 8km$$

$$\textit{Required APO} = \frac{2}{8+2} = 20\%$$

Since the required APO is greater than the available APO, passing lanes are warranted.

Lane Frequency:

$$LF = QFD + PLL = 8 + 2 = 10 \text{ km}$$

Check for Obsolescence:

Lane obsolescence volume, V_l, is given by:

$$V_l = 600 - 10.6s = 600 - 10.6(12) = 472 \text{ v/h}$$

Since the prevailing advancing volume of 360 v/h is less than the lane obsolescence volume, there is no condition of lane obsolescence.

The weighted average of the new APO (over a QFD + PLL = 10 km distance):

$$APO = \frac{8(16.1) + 2(100)}{8+2} = 32.9\%$$

No.3 Operational Analysis of a Two-Lane Highway with Existing Climbing Lanes

Given:
- 36km section of 2 lane highway
- 2, 1500m climbing lanes
  1. 4km into the section
  2. 34.5km into the section
- 14% trucks
- DHV(1992) 600vph
- DHV(2012) 750vph
- Directional Split 50/50
- Operating Speed 70km/h
- Speed Differential 20km/h
- POSD = 40% (NPZ = 60%)

Required: To provide adequate passing opportunity throughout the section.

Solution: Use an acceptable queue length of 6 vehicles.

HF = 28% (Fig B4-3)

Available APO_{1992} = (100 - NPZ)HF

= (100 - 60)(0.28) = 11.2%

$$\textit{Required APO} = \frac{PLL}{QFD+PLL}$$

$$QFD = \frac{QS^2}{V_{adv}(\Delta S)}$$

$$= \frac{6(70)^2}{300(20)} = 4.9km$$

$$use\ PLL = 2km$$

$$APO = \frac{2}{5+2} = 29\%$$

Available APO_{2012} = (100 - 60)(0.21) = 8.4%

Required APO

$$QFD = \frac{6(70)^2}{375(20)} = 3.9km$$

$$APO = \frac{2}{4+2} = 33.3\%$$

If we design for the 2012 requirements, the current needs will be met.

Lane Frequency
LF = QFD + PLL
= 4 + 2 = 6.0km

Divide road into six sections and calculate the weighted average of APO for Sections I and VI considering the presence of the truck climbing lanes:

$$APO = \frac{P_1W_1 + P_2W_2 + \ldots P_nW_n}{W_1 + W_2 + \ldots W_n}$$

$$= \frac{8.4(4)+100(1.5)}{4+1.5} = 33.4\%$$

Calculate the APO_{WA} for sections, II, III IV and V with passing lane provision

Sections II and V:

$$APO = \frac{8.4(4.5)+100(2)}{4.5+2} = 36.6\%$$

Sections III and IV:

$$APO = \frac{8.4(4)+100(2)}{4+2} = 38.9\%$$

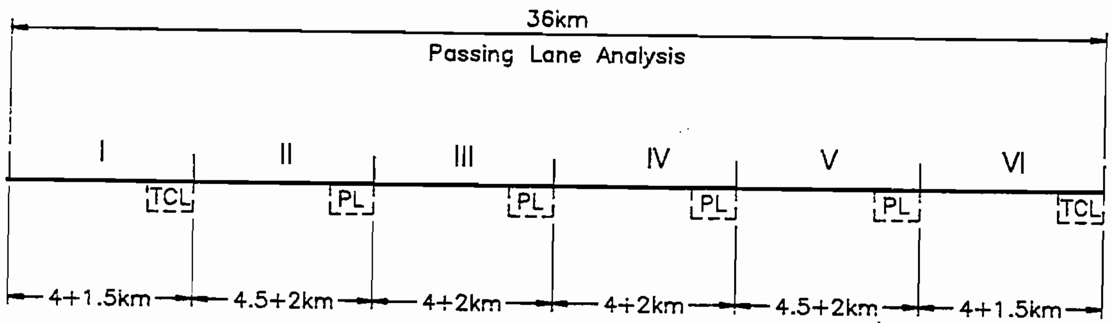
Check for lane obsolescence:

$$V_l = 600 - 10.6s$$
$$= 600 - 10.6(14) = 452 v/h$$

Since the 2012 volume is less than this, there is no condition of lane obsolescence.

The above spacing will meet the requirements of the design year and under current conditions it will provide:

Sections I and VI	APO = 35.4%
Sections II and V	APO = 38.5%
Sections III and IV	APO = 40.8%.



B.5 FREEWAYS

A freeway is a divided highway having at least 2 lanes in each direction of travel and full control of access. There are no external interruptions to traffic such as intersections or entrances to adjacent development. Access to the facility is through interchange ramps, generally providing for high-speed merging and diverging.

Operating conditions on freeways result primary from interaction between vehicles, and between vehicles and the geometric features of the highway.

Freeways consist of the following basic components from the point of view of capacity and level of service:

- Basic freeway segments - sections of freeway that are unaffected by weaving, merging or diverging manoeuvres. A basic freeway segment is outside the limits of 150 m upstream of a merge or downstream of a diverge and 750 m downstream of a merge or upstream of a diverge illustrated in Figure B5-1.
- Weaving sections - sections in which two or more vehicle flows cross and are sufficiently short so that the weaving manoeuvre affects the operation of the freeway.
- Ramp terminals - sections of freeway where on-ramps and off-ramps are joining and leaving the freeway, causing some turbulence due to concentrations of merging and diverging vehicles.
- Ramps - the main body of the roadway that provides for movement between a freeway and a crossing road, between the ramp terminals.

Each of these components is dealt with from the point of view of capacity and level of service in the following sections.

B.5.1 Basic Freeway Segments

Freeway capacity is defined as the maximum sustained flow at which traffic can pass a point, for a specified period of time (usually 15 min) under prevailing roadway and traffic conditions, expressed in vehicles per hour.
Roadway conditions referred to are the geometric features of the freeway and include such items as number and width of lanes, horizontal alignment, vertical alignment and lateral clearances.

Traffic conditions refer to characteristics of the traffic stream that affect capacity and operation, and include such features as lane distribution traffic mix and driver characteristics.

Ideal conditions for a freeway refer to geometric and environmental features and include the following:

- All lanes 3.75 m wide.
- Minimum lateral clearance between edge of travelled lanes and roadside and median obstacles, 2 m.
- All passenger cars.
- Drivers typical of weekday commuter traffic or other regular familiar drivers.
- Level terrain.

The capacity of a freeway under ideal conditions depends on the design speed as illustrated in Table B5-1.

Table B5-1
CAPACITY OF BASIC FREEWAY SEGMENTS

Design Speed km/h	Capacity, c pc/h/lane
120	2000
110	2000
100	2000
90	1950
80	1900

Flow, density and speed characteristics for ideal conditions are illustrated in Fig. B5-2 and B5-3. These relationships will vary slightly with posted speed limit but the general shape will remain the same. It will be noted that average speed varies very little with v/c ratio in the higher levels of service, but drops off sharply as the v/c ratio approaches 1.0 and the freeway approaches capacity. On the other hand, density increases steadily as the v/c ratio increases, rising more rapidly as the freeway comes close to capacity.

It is because of these characteristics that density rather than speed is chosen to define the boundaries of level of service. Speed is an important feature in the mind of the driver in expressing quality of service. Others such as freedom to manoeuvre and proximity to other vehicles are also important and these are directly related to density.

Density criteria for defining levels of service are given in Table B5-2.

For freeways, level of service is a qualitative measure of operational conditions by which drivers and passengers perceive their freedom to manoeuvre, comfort and service. These conditions are stated in terms of average speed, density and flow, with density being the most significant. The levels of service are defined in B.3.2.1.

Maximum service flow is the maximum hourly rate at which vehicles can reasonably be expected to pass a point of a lane or roadway during a given time period under prevailing roadway and traffic conditions while maintaining a designated level of service. Service flow is normally based on a 15 min time period.

Level of service criteria for basic freeway segments are defined in terms of density. Density is a measure of the proximity of vehicles in the traffic stream to each other and expresses the degree of manoeuvrability within the traffic stream. Limiting values for each level of service are given in Table B5-2.

The general expression for determining service flow for a given level of service for traffic operations on general terrain basic freeway segments is:

$$SF_i = c \times N \times (v/c)_i \times f_w \times f_{hv} \times f_p$$

where:

SF_i = service flow; the maximum flow that can be accommodated by the freeway segment in one direction, under prevailing roadway and traffic conditions, while meeting the performance criteria of LOSi, in v/h.

c = capacity under ideal conditions taken from Table B5-1.

N = number of lanes in one direction of the freeway.

$(v/c)_i$ = maximum volume to capacity ratio for level of service i, taken from Table B5-2.

f_w = factor to adjust for the effects of restricted lane widths and/or lateral clearances; taken from Table B5-3.

f_{hv} = factor to adjust for the effect of heavy vehicles (trucks, buses and recreational vehicles) in the traffic stream, determined as described below.

f_p = factor to adjust for the effect of driver population, determined from Table B5-5.

The factor to adjust for heavy vehicles is found by first determining the passenger car equivalent, E, for trucks, buses and recreational vehicles in the traffic stream. For extended general freeway segments, values are taken from Table B5-4. The heavy vehicle factor f_{hv} is then calculated by the expression:

$$^{*}f_{hv} = \frac{1}{1 + P_t(E_t-1) + P_b(E_b-1) + P_r(E_r-1)}$$

* If only one type of heavy vehicle is present, f_{hv} can be found from Table 3-9 in the Highway Capacity Manual 1985.

where:

P_t = proportion of trucks in traffic stream, expressed as a decimal;

P_b = proportion of buses in the traffic stream, expressed as a decimal

P_r = proportion of RV's in the traffic stream, expressed as a decimal;

E_t = passenger-car equivalent for trucks, obtained from Table B5-3;

E_b = passenger-car equivalent for buses, obtained from Table B5-3;

E_r = passenger-car equivalent for RV's, obtained from Table B5-3

The adjustment factor f_p is used to reflect the influence of driver population. It is generally accepted that regular weekday drivers of the commuter type use freeways more efficiently than weekend and recreational drivers.

This can be accounted for in the selection of f_p and Table B5-5 provides some direction. Values in the table should be used with caution and local data should be used wherever it is available.

Table B5-2
LEVEL OF SERVICE CRITERIA AND V/C RATIOS FOR BASIC FREEWAY SEGMENTS

v/c

Level of Service	Density pc/km/lane	Design Speed km/h 120	110	100	90	80
A	≤ 7	0.38	0.34	-	-	-
B	≤ 12	0.57	0.53	0.50	-	-
C	≤ 19	0.80	0.74	0.70	0.68	0.67
D	≤ 26	0.97	0.89	0.86	0.84	0.83
E	≤ 42	1.00	1.00	1.00	1.00	1.00
F	> 42	a	a	a	a	a

a: Highly variable and unstable

Table B5-3

WIDTH FACTORS FOR RESTRICTED LANE WIDTH AND LATERAL CLEARANCE FOR FREEWAYS

DISTANCE FROM EDGE OF TRAVELLED WAY TO OBSTRUCTION (m)	WIDTH FACTOR, f_w									
	OBSTRUCTION ON ONE SIDE OF ROADWAY					OBSTRUCTION ON BOTH SIDES OF ROADWAY				
	LANE WIDTH (m)									
	3.75	3.50	3.25	3.00	2.75	3.75	3.50	3.25	3.00	2.75
4-LANE FREEWAY (2 LANES EACH DIRECTION)										
≥ 2.0	1.00	0.99	0.96	0.90	0.82	1.00	0.99	0.97	0.90	0.83
1.5	0.99	0.96	0.95	0.88	0.81	0.99	0.98	0.94	0.88	0.80
1.0	0.98	0.97	0.94	0.87	0.80	0.97	0.95	0.92	0.86	0.78
0.5	0.96	0.94	0.91	0.85	0.79	0.94	0.92	0.88	0.83	0.75
0.0	0.90	0.89	0.86	0.80	0.73	0.82	0.80	0.77	0.72	0.66
6 OR 8-LANE (3 LANES EACH DIRECTION)										
≥ 2.0	1.00	0.99	0.95	0.88	0.79	1.00	0.99	0.95	0.88	0.79
1.5	0.99	0.97	0.93	0.86	0.77	0.99	0.97	0.93	0.86	0.77
1.0	0.98	0.96	0.92	0.85	0.76	0.97	0.95	0.91	0.84	0.76
0.5	0.97	0.95	0.91	0.84	0.75	0.95	0.93	0.89	0.82	0.74
0.0	0.94	0.92	0.89	0.83	0.74	0.91	0.90	0.86	0.79	0.70

Table B5-4

PASSENGER CAR EQUIVALENTS ON EXTENDED BASIC FREEWAY SEGMENTS

	Terrain		
Factor	**Level**	**Rolling**	**Mountainous**
E_T (for trucks)	1.7	4.0	8.0
E_B (for buses)	1.5	3.0	5.0
E_R (for RVs)	1.6	3.0	4.0

Table B5-5
POPULATION FACTOR ADJUSTMENT

Traffic Type	Population Factor, f_p
Weekday/Commuter	1.0
Weekend/Recreational	0.75 - 0.90

B.5.2 Weaving

Weaving is defined as the crossing of two or more traffic streams travelling in the same general direction along a significant length of highway, without the aid of traffic control devices. Weaving areas are formed when a merge area is closely followed by a diverge area, or when an on-ramp is closely followed by an off-ramp and the two are joined by an auxiliary lane.

Weaving areas require intense lane-changing manoeuvre as drivers must access lanes that will direct them to the required downstream roadway. Thus, traffic in a weaving area is subject to turbulence in excess of that normally present on basic highway sections. This turbulence presents special operational problems and design requirements that are addressed by the procedures of this chapter.

Weaving areas may occur on any type of highway: freeways, multilane highways, two-lane highways, or arterials. They are most prevalent, however, as parts of freeway systems. The procedures of this chapter apply to freeway weaving areas, but may be applied as an approximation to other types of facilities.

B.5.2.1 Configuration

Configuration refers to the relative location and number of entrance lanes and exit lanes for each section, and can have a significant impact on the extent of lane-changing that must take place in the weaving section. The procedures deal with those primary type of weaving configurations referred to as Type A, Type B and Type C weaving sections. They are illustrated in Figures B.5-4, B.5-5 and B.5-6, and are described below:

Type A weaving areas require that each weaving vehicle make one lane change in order to execute the desired movement. Figure B.5.4 shows two examples of Type A weaving areas. An on-ramp is followed by an off-ramp, with a continuous auxiliary lane between the ramps. All on-ramp vehicles must make a lane change out of the auxiliary lane into the first lane of the freeway, and all off-ramp vehicles must make a lane change from the first lane of the freeway to the auxiliary lane. Lane changes to and from the outer lanes of the freeway may also take place within the section, but these are not mandated or required by the weaving movement.

Sections formed by on-ramp/off-ramp sequences joined by continuous auxiliary lanes are often referred to as ramp-weave sections. They may also be referred to as one-sided weaving sections, because all weaving movements take place on one side of the roadway. It should be noted that on-ramps followed by off-ramps that are not joined by a continuous auxiliary lane are not considered to be weaving areas. They are treated as separate merge and diverge areas and analyzed using the procedures of Section B5-3.

Because weaving vehicles in a Type A weaving area must cross the crown line, weaving vehicles are usually confined to occupying the two lanes adjacent to the crown line while in the weaving section. Normally, some non-weaving vehicles will also remain in lanes adjacent to the crown line. Lanes adjacent to the crown line are, therefore, generally shared by weaving and non-weaving vehicles. One of the most significant effects of configuration on operations is to limit the maximum number of lanes which weaving vehicles may occupy while travelling the section.

All weaving sections classified as Type B involve multilane entry and/or exit legs. Two critical characteristics distinguish Type B weaving areas from all others:

1. One weaving movement may be accomplished without making any lane changes.

2. The other weaving movement requires at most one lane change.

Figures B5-5 shows two such weaving areas. In both illustrations (a) and (b), movement B-C can be made without executing any lane changes, while movement A-D requires only one lane change. In (a), this is accomplished by providing a diverging lane at the exit. From this lane, a vehicle may proceed on either exit leg without making a lane change. This type of design is also referred to as lane balanced, that is, the number of lanes leaving the diverge is one greater than the number of lanes approaching it. In (b), a lane from leg A is merged with a lane from leg B at the entrance gore area.

Type B weaving areas are extremely efficient in carrying large weaving volumes, primarily because of the provision of a "through lane" for one of the weaving movements. Weaving manoeuvres can be accomplished with a single lane change from the lane or lanes adjacent to this "through lane." Thus, weaving vehicles can occupy a substantial number of lanes in the weaving section, and are not as restricted in this regard as in Type A sections.

Illustration (c) shows an unusual configuration in which both a merge of two lanes at the entrance gore and lane balance at the exit gore are provided. In this case, both weaving movements can be made without a lane change. Weaving movements can be made with a single lane change from the two lanes adjacent to the "through lane". Such configurations are usually found on collector-distributor roadways. While some weaving movements are accomplished as a merge followed by a diverge, lane changes to and from lanes adjacent to the "through lane" yield real weaving activity, and these sections are analyzed as weaving areas.

Type C weaving areas are similar to Type B sections in that one or more "through lanes" are provided for one of the weaving movements. The distinguishing feature between Type B and Type C sections is the number of lane changes required for the other weaving movement. A Type C weaving area is characterized by:

1. One weaving movement may be accomplished without making a lane change.

2. The other weaving movement requires two or more lane changes.

Figure B5-6 shows two Type C weaving areas. In (a), movement B-C does not require lane-changing, while movement A-D requires two lane changes. This type of section is formed when there is neither a merging of lanes at the entrance gore nor lane balance at the exit gore and no crown line exists. While such a section is relatively efficient for weaving movements in the direction of the "through lane," it cannot efficientlyhandle large weaving volumes in the other direction.

Illustration (b) shows a two-sided weaving area. It is formed when a right-hand on-ramp is followed by a left-hand off-ramp or vice-versa. In such cases, the through volume on the freeway is functionally a weaving movement. Ramp-to-ramp vehicles must cross all lanes of the freeway to execute their desired manoeuvre. Freeway lanes are, in effect, through weaving lanes. Ramp-to-ramp drivers must execute three lane changes in illustration (b). Although technically a Type C configuration, there is little information concerning the operation of such sections, and the methodology of this chapter is only an approximation of their characteristics. They should generally be avoided in cases where there is any significant ramp-to-ramp volume.

Weaving configuration is determined by the basic number of required lane changes that must be made by the two weaving movements in the notion, illustrated in Table B5-6.

B.5.2.2. Constrained and Unconstrained Weaving

In general, vehicles in a weaving area will make use of available lanes in such a way that all component flows achieve approximately the same average running speed, with weaving flows somewhat slower than non-weaving flows. Occasionally, the configuration limits the ability of weaving vehicles to occupy the proportion of available lanes required to achieve this equivalent or balanced operation. In such cases, weaving vehicles occupy a smaller proportion of the available lanes than desired, while non-weaving vehicles occupy a larger proportion of lanes than for balanced operation. When this occurs, the operation of the weaving area is classified as constrained by the configuration. The result of constrained operation is that non-weaving vehicles will operate at significantly higher speeds than weaving vehicles.

Where configuration does not restrain weaving vehicles from occupying a balanced proportion of available lanes, the operation is classified as unconstrained. Average running speeds of weaving and non-weaving vehicles generally differ by less than 10 km/h except in short Type A sections, where acceleration and deceleration of ramp vehicles limit their average speed regardless of the use of available lanes.

A major component of the procedure is the determination of whether operations in a given section are constrained or unconstrained.

B.5.2.3 Weaving Section Analysis

The following procedure provides for the analysis of simple weaving sections to determine the level of service for a given set of conditions. Roadway and traffic information is required including weaving length, type of configuration, number of lanes, lane widths, terrain or grade, weaving and non-weaving flow by movement, the peak hour factor, and traffic composition.

The weaving parameters used in this procedure, and their symbols are given in Table B5-7.

Weaving analysis is made easier through the use of a weaving diagram. A weaving diagram is a schematic drawing showing weaving and non-weaving flows in a weaving area. Step 3 of the example calculation shows the construction of such a diagram. Note that the weaving diagram illustrates actual flows in a straight-line form. The relative placement of entrance and exit points (A, B, C, D) in the diagram matches the actual site to ensure proper placement of weaving and non-weaving flows relative to each other. Flows on the weaving diagram should represent flow for the peak 15 min under ideal conditions, expressed in pc/h. It is also convenient to use the weaving diagram as a guide in computing the parameters used during the analysis.

The method of measuring weaving length is illustrated in Figure B5-7.

Evaluation of the level of service in an existing or projected weaving area is accomplished using the following computational steps:

1. Establish roadway and traffic conditions.
2. Convert all traffic volumes to peak flows under ideal conditions.
3. Construct weaving diagram.
4. Calculate unconstrained weaving and non-weaving speeds.
5. Check for constrained operation.
6. Check for weaving area limitations.
7. Determine level of service.

Table B5-6

LANE CHANGES BY CONFIGURATION TYPE

Minimum Number of Required Lane Changes for One Movement	**Minimum Number of Required Lane Changes for the Other Movement**		
	0	**1**	**2**
0	Type B	Type B	Type C
1	Type B	Type A	-
2	Type C	-	-

The following example calculation shows each of the seven steps noted above and illustrates the procedure. The example is for a Type B weaving section. For other types, the same procedure is used but with different constants in Table B5-8 for calculating weaving and non-weaving speeds, different expressions in Table B5-9 for calculating the number of lanes required for unconstrained operation, and different values are used in Table B5-10, for limitations on weaving sections.

Example Analysis

1. Roadway and Traffic Conditions

Referring to Figure B5-7, the traffic volumes are:

AC 1810 v/h
AD 690 v/h
BC 1030 v/h
BD 1290 v/h

Trucks: 7%
Buses: 0%
RV's: 0%
PHF: 0.91
Weaving length: 460 m
Terrain: level
Number of Lanes, (N): 4
Lane width: 3.75 m
Lateral obstructions: none
Driver population: commuters
Required: To determine level of service at which the section will operate. *

Table B5-7

WEAVING PARAMETERS

SYMBOL	PARAMETER
L	Length of weaving area, in m
L_H	Length of weaving area, in hundreds of m
N	Total number of lanes in the weaving area
N_W	Number of lanes used by weaving vehicles in the weaving area
N_{nw}	Number of lanes used by nonweaving vehicles in the weaving area
v	Total flow in the weaving area, in passenger car equivalents, in pc/h
v_w	Total weaving flow in the weaving area, in passenger car equivalents, in pc/h
v_{w1}	Weaving flow for the larger of the two weaving flows, in passenger car equivalents, in pc/h
v_{w2}	Weaving flow for the smaller of the two weaving flows, in passenger car equivalents, in pc/h
v_{nw}	Total nonweaving flow in the weaving area, in passenger car equivalents, in pc/h
VR	Volume ratio v_w/v
R	Weaving ratio v_{w2}/v_w
S_w	Average running speed of weaving vehicles in the weaving area, in km/h
S_{nw}	Average running speed of nonweaving vehicles in the weaving area, in km/h8

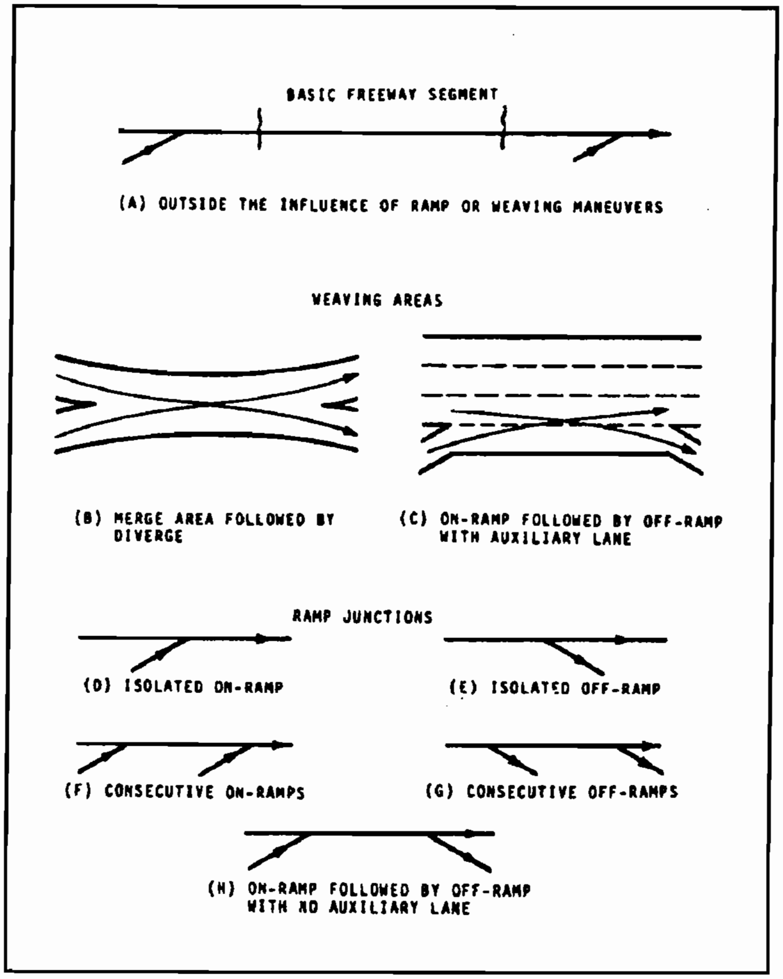


Figure B5-1

Freeway Components

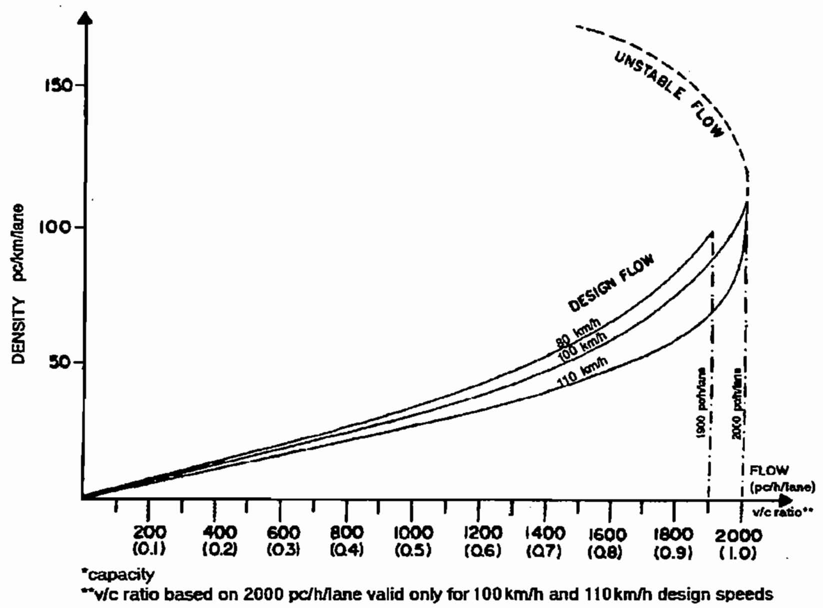
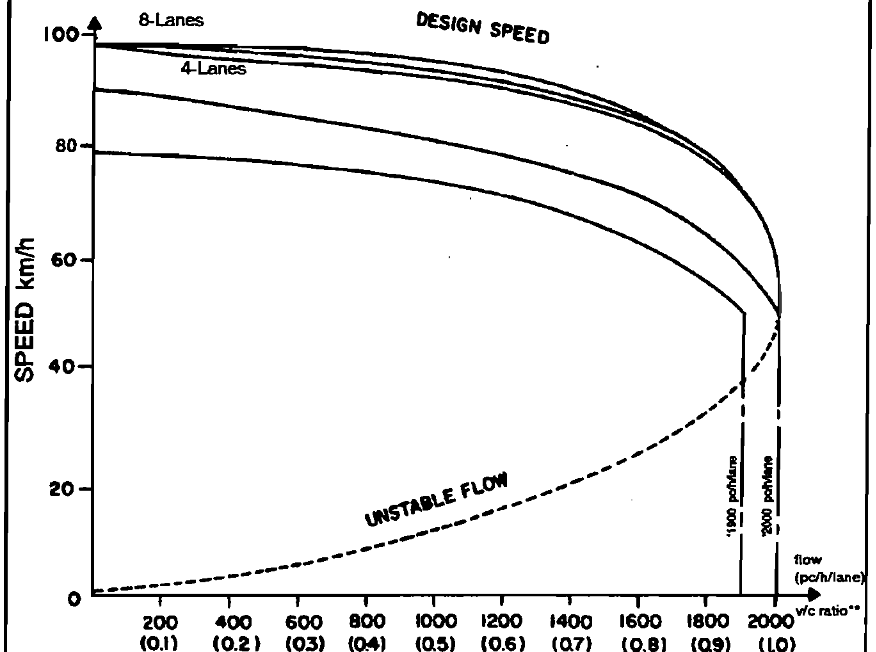


Figure B5-2

Density - Flow Relationship for Freeways



*capacity
**v/c ratio based on 2000 pc/h/lane valid only for 100 km/h and 110 km/h design speeds

Figure B5-3

Speed - Flow Relationship for Freeways

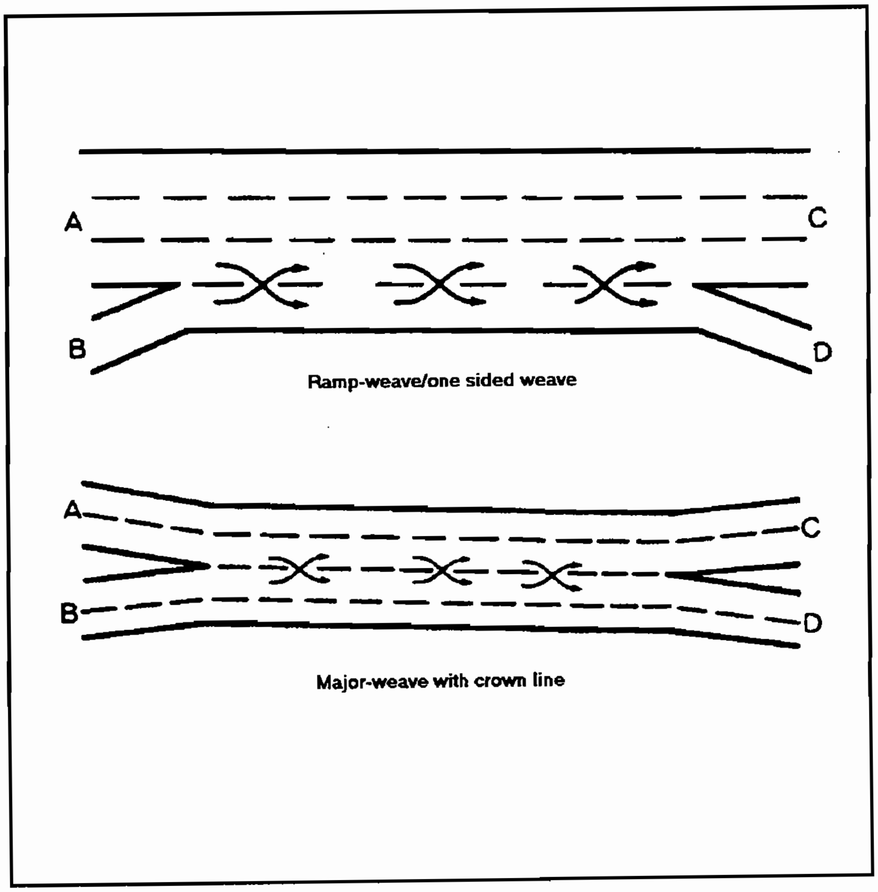


Figure B5-4

Type A Weaving Sections

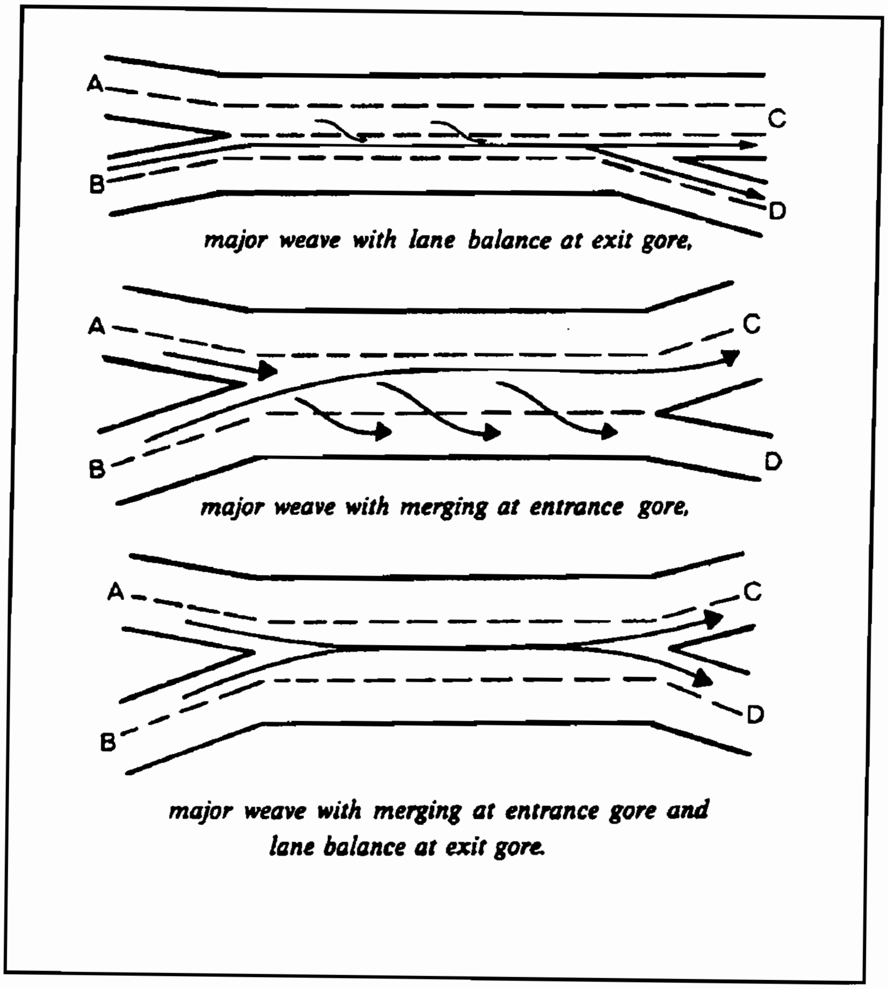


Figure B5-5

Type B Weaving Sections

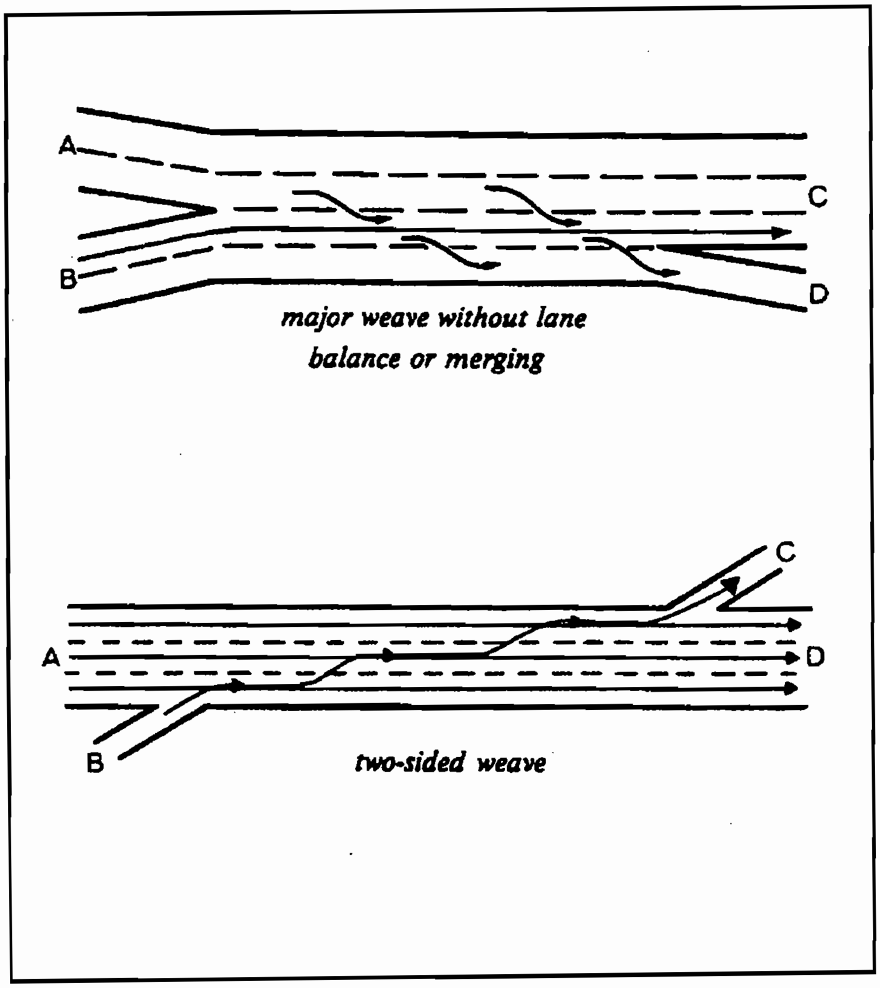


Figure B5-6

Type C Weaving Sections

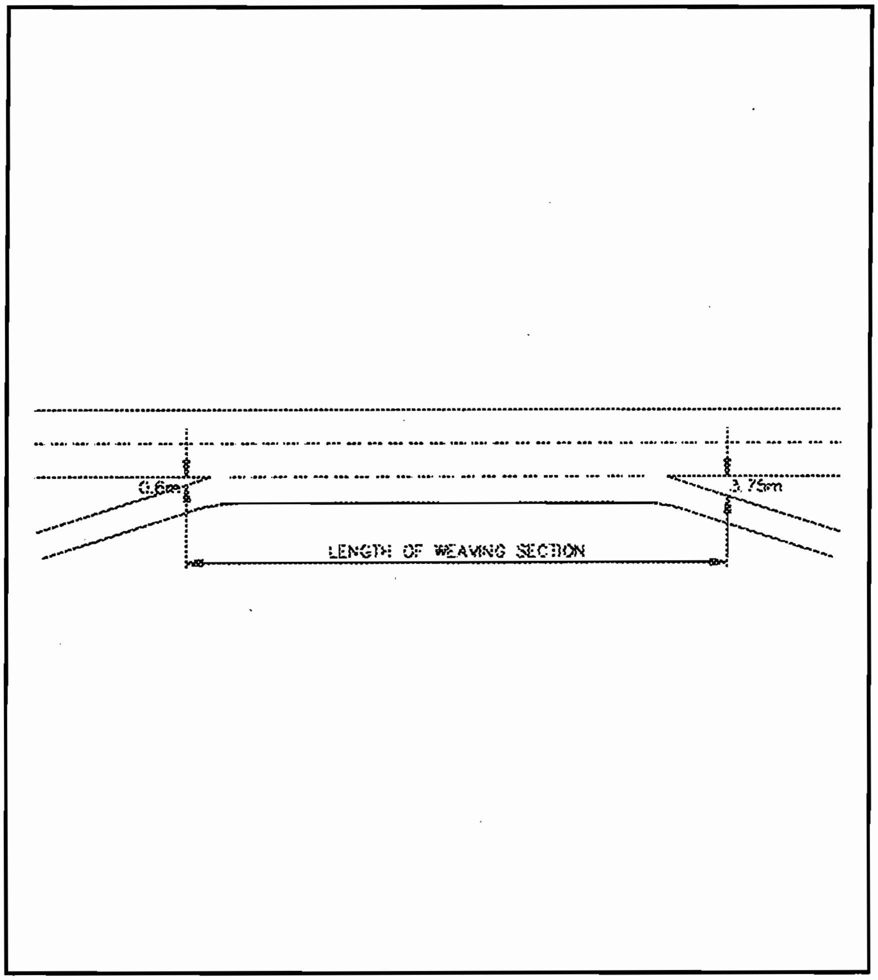


Figure B5-7

Measurement of Weaving Length

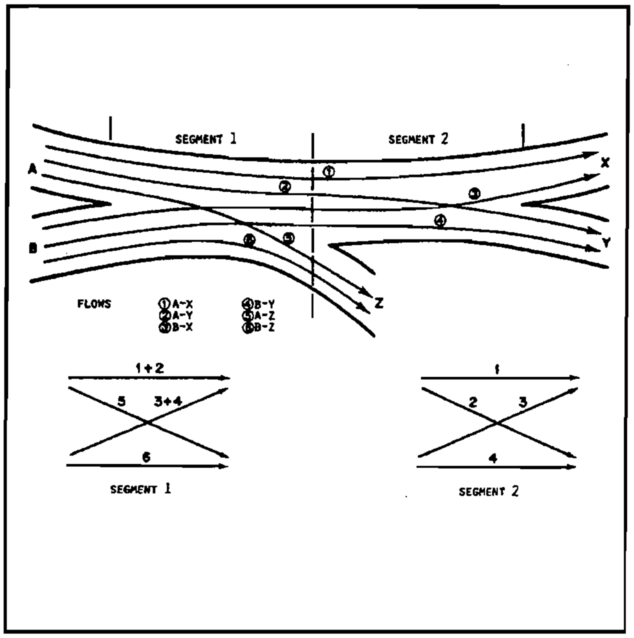


Figure B5-8
Multiple Weaving:
Merge Followed by Two Diverges

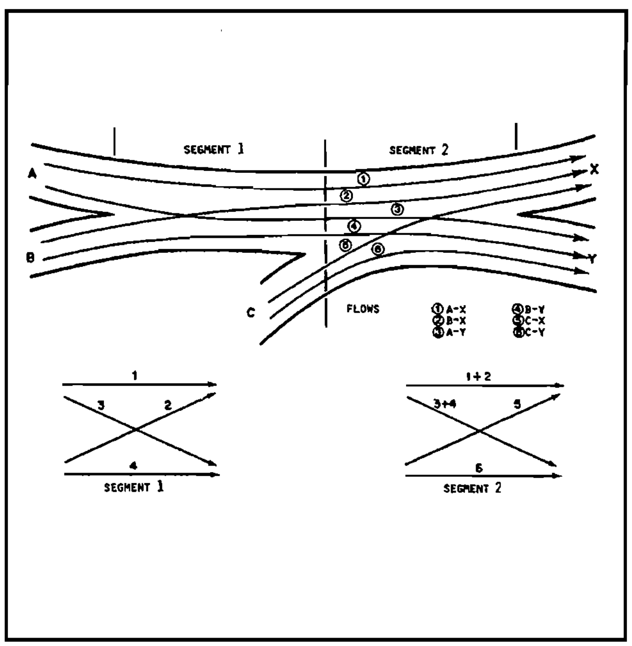


Figure B5-9
Multiple Weaving:
Two Merges Followed by a Diverge

Table B5-8

CONSTANTS FOR CALCULATING WEAVING AND NON-WEAVING SPEEDS

$$S_w \text{ or } S_{nw} = 24 + \frac{80}{1 + a(1 + FR)^b (v/N)^c / (3.28\ L)^d} \text{ km/h}$$

Type of Configuration	Constants for Weaving Speed, S_w a	b	c	d	Constants for Non-weaving SPEED, S_{nw} a	b	c	d
TYPE A								
Unonstrained	0.226	2.2	1.00	0.90	0.020	4.0	1.30	1.00
Constrained	0.280	2.2	1.00	0.90	0.020	4.0	0.88	0.60
TYPE B								
Unconstrained	0.100	1.2	0.77	0.50	0.020	2.0	1.42	0.95
Constrained	0.160	1.2	0.77	0.50	0.015	2.0	1.30	0.90
TYPE C								
Unconstrained	0.100	1.8	0.80	0.50	0.015	1.8	1.10	0.50
Constrained	0.100	2.0	0.85	0.50	0.013	1.6	1.00	0.50

Table B5-9

CRITERIA FOR UNCONSTRAINED WEAVING

Configuration	No. of Lanes Required for Unconstrained Operation, N_w	Maximum No. of Weaving Lanes, N_w(max)
A	$1.212N(FR)^{0.571}L^{0.234}/S^{0.438}$	1.4
B	$N[0.085 + 0.703FR + (71.6/L) - 0.0112(S_{nw} - S_w)]$	3.5
C	$N[0.761 - 0.00036L - 0.0031(S_{nw} - S_w) + 0.047FR]$	3.0*

* For 2-sided weaving, all freeway lanes may be used for weaving.

Table B5-10

LIMITATIONS ON WEAVING SECTIONS

	Maximum Values				
Configuration	Total Weaving Flow, v_w pc/h	Flow Per Lane v/N pc/h/lane	Flow Ratio FR	Weaving Ratio R	Weaving Length L m
A	1800	1900	N FR 2 1.00 3 0.45 4 0.35 5 0.22	0.50	600
B	3000	1900	0.80	0.50	760
C	3000	1900	0.50	0.40	760

2. Convert Traffic Volumes to Traffic Flows

$$v = \frac{V}{PHF \times f_{hv} \times f_w \times f_p}$$

where v is traffic flow in pc/h

V is traffic volume in v/h

f_w is width factor from Table B5-3

f_p is population factor from Table B5-5

PHF = 0.91

E_t = 1.7 (from Table B5-4)

$$f_{hv} = \frac{1}{1 + P_t(E_t-1)}$$

$$= \frac{1}{1 + 0.07\ (1.7 - 1)}$$

$$= 0.95$$

f_w = 1.0

f_p = 1.0

$$v = \frac{V}{0.91 \times 0.95 \times 1 \times 1}$$

$$= \frac{V}{0.86}$$

Flows:

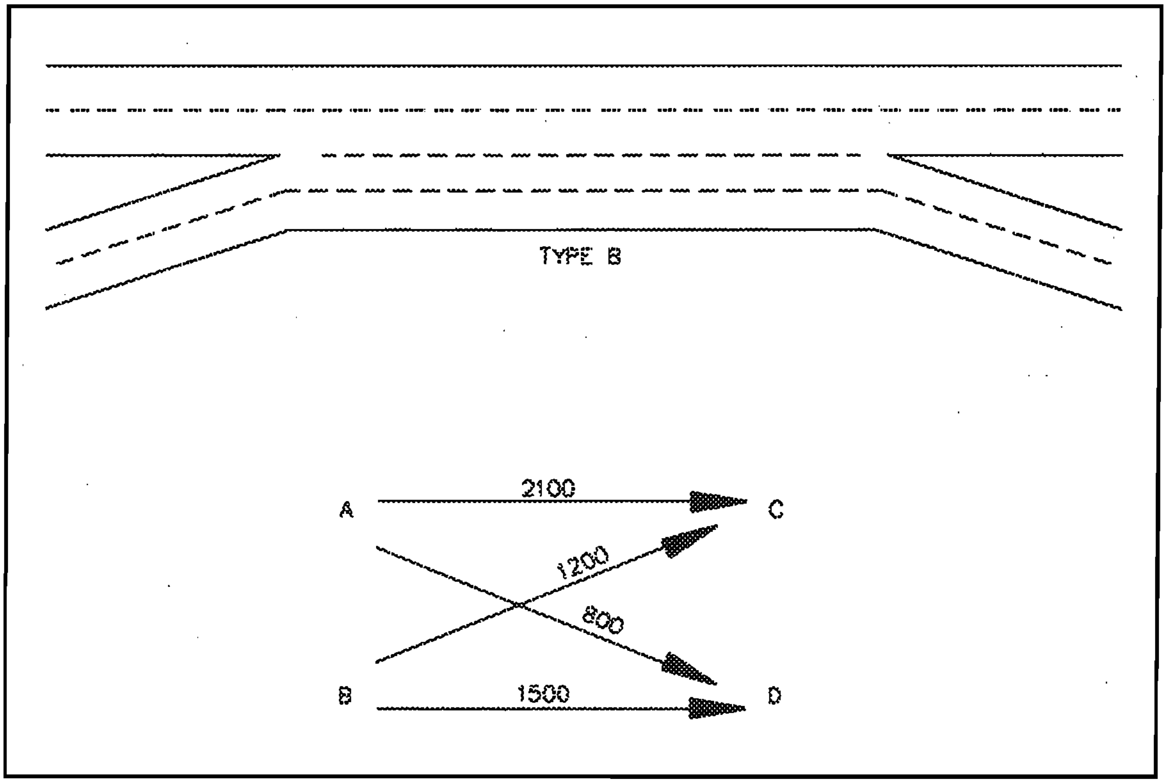
AC: 1810/0.86 = 2100 pc/h

AD: 690/0.86 = 800 pc/h

BC: 1030/0.86 = 1200 pc/h

BD: 1290/0.86 = 1500 pc/h

3. Construct Weaving Diagram



4. Calculate Unconstrained Weaving and Non-Weaving Speeds

Type B Weaving
Weaving flow, $v_w = 1200 + 800 = 2000$
Total flow, $v = 2000 + 2100 + 1500 = 5600$
N = total number of lanes in weaving section; 4

Weaving ratio,

$$R = \frac{\text{smaller weaving flow}}{\text{total weaving flow}} = \frac{800}{2000} = 0.400$$

Flow ratio,

$$FR = \frac{\text{Weaving flow}}{\text{total flow}} = \frac{2000}{5600} = 0.357$$

$$v/N = \frac{5600}{4} = 1400$$

Unconstrained weaving speed,

$$S_w = 24 + \frac{80}{1 + a(1 + FR)^b (v/N)^c/(3.28L)^d}$$

L = length of weaving section; 460 m

From Table B5-8:
a = 0.100
b = 1.2
c = 0.77
d = 0.50

S_w

$$= 24 + \frac{80}{1+0.1(1+0.357)^{1.2}(1400)^{0.77}/(3.28x460)^{0.50}}$$

= 64.35 or 64 km/h (rounded)

Unconstrained non-weaving speed,

$$S_{nw} = 24 + \frac{80}{1 + a(1 + FR)^b (v/N)^c / (3.28L)^d}$$

N and L are as above

From Table B5-8

a = 0.020
b = 2.0
c = 1.42
d = 0.95

$$S_{nw} = 24 + \frac{80}{1 + 0.020(1+0.357)^{2.0}(5600/4)^{1.42}/(3.28 \times 460)^{0.95}}$$

= 63.36 or 63 km/h (rounded).

5. Check for Constrained Operation

Table B5-9 gives expressions to calculate the number of lanes required for unconstrained operation; N_w, and the maximum number of weaving lanes N_w (max) for each configuration type. Using the speeds for unconstrained weaving and non-weaving, calculated in step 4, the number of lanes N_w is calculated.

From Table B5-9

N_w = N(0.085+0.703FR+(71.6/L)-0.0112(S_{nw}-S_w)

= 4(0.085 + 0.703 x 0.357 + (71.6/460) -

0.0112(63 - 64))

= 2.01

Since this is less than 3.5 shown in Table B5-9, the section will operate in the unconstrained mode.

If this calculation had shown N_w greater than 3.5, the section would then operate in the constrained mode. The weaving and non-weaving speeds would then have to be re-calculated using the parameters for constrained operation in Table B5-8 before proceeding to step 6.

6. Check For Weaving Area Limitations

Table B5-9 gives limitations on weaving section equations. The values in the example are compared to confirm that they are below the values in the table.

Type B	**Example Calculation**	**Max. Value From Table B5-10 for Type B**
Weaving flow, v_w	2000 pc/h	3000 pc/h
Flow/lane, v/N	1400 pc/h	1900 pc/h
Flow ratio, FR	0.357	0.80
Weaving ratio, R	0.400	0.50
Weaving length, L	460 m	760 m

If any of the calculated values are higher than the maximum values from Table B5-9, lower quality operation than predicted by the calculations can be expected. If the procedure is used for design, consideration should be given to increasing the number of weaving lanes, the length of the weaving section or grade separation for weaving movements.

7. Determine Level of Service

Levels of service in weaving areas are directly related to weaving and non-weaving speeds. The criteria are given in Table B5-11.

From the calculations above:

Weaving speed is 64 km/h
Non-weaving speed is 63 km/h

Table B5-11

LEVEL OF SERVICE CRITERIA FOR WEAVING SECTIONS

Level of Service	Minimum Ave. Weaving Speed for Level Shown	Minimum Ave. Non-Weaving Speed for Level Shown
A	88	97
B	80	87
C	72	77
D	64	68
E	56-48*	56-48*
F	56-48*	56-48*

* The 56 value for LOS E/F is used when comparing with calculated speed using Table B5-8. The 48 value is used for comparison withfield measured speeds.

From Table B5-11, the level of service for weaving traffic is D and the level of service for non-weaving traffic is E.

B.5.2.4 Multiple Sections Weaving

Multiple weaving areas occur when one merge point is followed closely by two diverge points, or where two merge points are closely followed by a single diverge point. In such cases, several sets of weaving movements take place over the same segments of freeway, and lane-changing turbulence may be higher than that found in simple weaving areas.

Drivers will carefully select where to execute their required lane changes in a manner that minimizes interference with other weaving movements. Figure B5-8 and B5-9 show the two types of multiple weaving conditions, and where weaving movements are assumed to take place. This results in the formation of weaving diagrams for each subsegment of the weaving area, each of which can be analyzed as a simple weaving area using the procedure described above.

B.5.3 Ramps and Ramp Terminals

B.5.3.1 Ramp Components

A ramp is a length of roadway providing for travel between two grade-separated highways, of which one is normally a freeway. It may be regarded as consisting of three types of element:

- A ramp-freeway terminal
- A ramp roadway
- A ramp-street terminal

A ramp-freeway terminal is generally designed to permit high-speed merging or diverging movements to take place with a minimum of disruption to the adjacent freeway traffic stream. The geometric characteristics of ramp-freeway terminals vary. Elements such as the provision and length of acceleration/deceleration lanes, angle of convergence or divergence, relative grades on the freeway and ramp, and other aspects influence ramp operations. Although the procedures of this section are primarily applicable to high-type designs, many of the relationships used were calibrated using data from a variety of geometric cases, including some which may be regarded as substandard. Thus, these relationships can be applied to cases with less than ideal geometrics, as noted in the procedures. Geometric design standards for ramps and ramp terminals are given in Chapter F.

The ramp roadway itself may also vary widely from location to location. Ramps vary in the number of lanes (usually one or two), length, design speed, grades, and horizontal curvature. The ramp roadway itself is rarely a source of operational difficulties, unless a traffic incident causes a disruption along its length.

The ramp-street terminal can be of a type permitting uncontrolled merging or diverging movements to take place, or it can take the form of an at-grade intersection.

This section provides procedures for the capacity analysis of ramp-freeway terminals and ramp roadways. At-grade intersections may be analyzed using the procedures of sections B6 and B7.

B.5.3.2 Operational Characteristics

A ramp-freeway terminal is an area of competing traffic demands for space. Upstream freeway demand competes with on-ramp demand in merge areas. On-ramp demand is usually generated locally, although collector and arterial streets may bring vehicles to the ramp from more distant origins. The freeway flow upstream of an on-ramp is the composite of upstream demands from a variety of sources.

In this chapter, lanes are numbered from 1 to N, from the shoulder to the median.

In the merge area, on-ramp vehicles try to find openings, or gaps, in the adjacent freeway lane traffic stream. As most ramps are on the right side of the facility, freeway lane no. 1 is most affected.

As the on-ramp flow increases, the entering vehicles influence the distribution of traffic on the freeway lanes as traffic shifts to avoid the turbulence and conflicts in the merging area. The situation is a dynamic one in which the flows interact, with the on-ramp flow generally having a significant influence on overall operations. In the relationships used, the on-ramp volume is specified independently, and the lane 1 volume is thought of as being dependent on it as well as on other variables.

At off-ramps, the basic manoeuvre is a diverge. Exiting vehicles must occupy the lane adjacent to the ramp (or dedicated to the ramp exit), so that there is a net effect of other drivers redistributing themselves amongst the other lanes. Where two-lane off-ramps are present, the influence of diverging movements may spread over several lanes of the freeway.

Procedures in this section treat the freeway and ramp volumes as inputs to a ramp capacity analysis, with the level of service as the output or result of the analysis.

B.5.3.3 Ramp Configuration

As the characteristics of adjacent upstream and downstream ramps influence the operations at any given location, ramp analysis must consider ramp sequences rather than each ramp in an isolated fashion. To avoid treating an unreasonable number of different configurations, ramps are generally examined in pairs. Thus, where a ramp has both adjacent upstream and downstream ramps close enough to impact its operation, it will generally be considered twice - one in conjunction with the upstream ramp, and then in conjunction with the downstream ramp. This is discussed in Analysis Procedures, B.5.3.6.

This section specifically addresses the following ramp configurations, illustrated in Figure B5-10:

1. Isolated on-ramp - An on-ramp with no adjacent ramp close enough to influence its operations. The term, "close enough", varies, depending on volumes and other factors; however, ramp spacings greater than 1800 m are always considered beyond the range of influence.

2. Isolated off-ramp - An off-ramp with no adjacent ramp close enough to influence its operations.

3. Adjacent on-ramps - Two consecutive on-ramps close enough to mutually influence their behaviour.

4. Adjacent off-ramps - Two consecutive off-ramps close enough to mutually influence their behaviour.

5. On-ramp followed by off-ramp - An on-ramp, off-ramp sequence spaced closely enough to mutually influence each other's behaviour. If the ramps are joined by a continuous auxiliary lane, the section is treated as a ramp-weave area and analyzed using the procedures of section B.5.2; if no auxiliary lane is present, the procedures in this section are used.

6. Off-ramp followed by on-ramp - An off-ramp, on-ramp sequence spaced closely enough to mutually influence each other's behaviour. Such a ramp sequence often operates as if the ramps were isolated.

7. Lane additions - A one-lane on-ramp that is continuous with an additional freeway lane downstream.

8. Lane drops - A one-lane off-ramp that results in the deletion of one freeway lane at the ramp-freeway junction.

9. Major diverge points - The separation of a freeway segment into two multilane freeway or collector/distributor roadways. This applies only to those configurations for which the total number of lanes departing the diverge point is equal to the number of lanes approaching it plus one.

10. Major merge point - The joining of two multilane freeway or collector/distributor roadways into a single freeway segment. This applies only to configurations in which two approach lanes (one from each approach) are merged into a single lane.

11. Two lane ramps - Two-lane on-ramps or off-ramps where there are no lane additions or drops at the ramp-freeway terminal (not illustrated).

B.5.3.4 Critical Components for Analysis

The critical components for ramp configurations are:

1. Merge volume, V_m - This term applies to on-ramps and is the total volume in the traffic streams that will join. For the case of a one-lane, right-side on-ramp, the merge volume is the sum of the lane 1 volume plus the ramp volume.

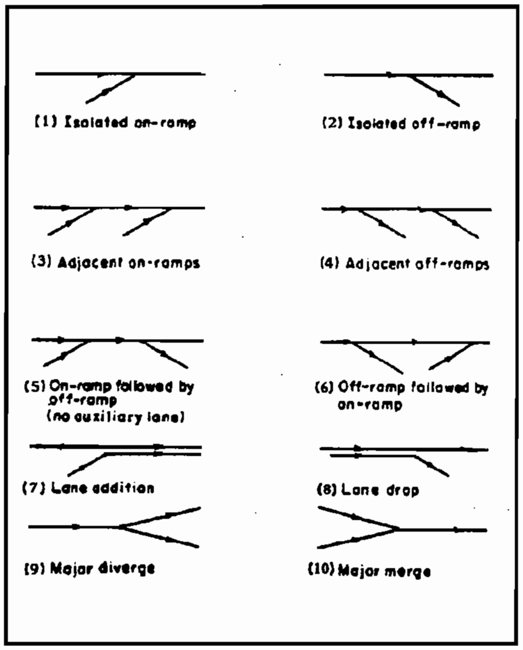


Figure B5-10

Ramp Configurations

2. Diverge volume, V_d - This term applies to off- ramps. It is the total volume in the traffic stream which will separate. For the case of a one-lane, right-side off-ramp, the diverge volume is equal to the lane 1 volume immediately upstream of the subject ramp.

3. Freeway volume, V_f - At any merge or diverge location, the total freeway volume must also be considered. The freeway volume is generally considered at the point where it is at the maximum level, i.e., upstream of an off-ramp and downstream of an on-ramp.

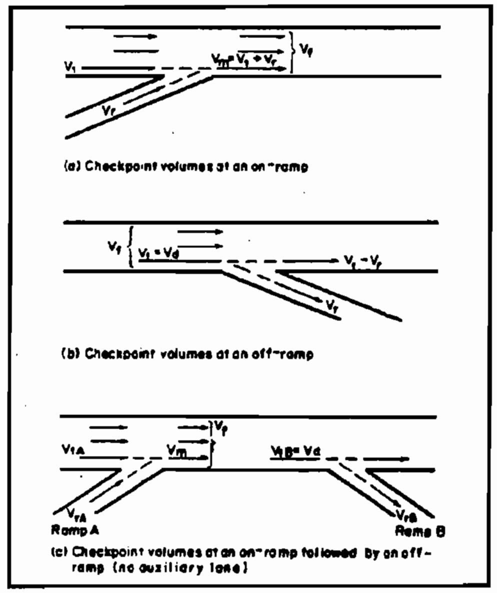


Figure B5-11

Checkpoint Volumes for Ramp-Freeway Terminals

Figure B5-11 shows the relationships of these critical volumes and other volume elements. The merge, diverge, and freeway volumes are often referred to as "checkpoint" volumes, as it is these values to which level-of-service criteria are applied.

The critical components of the traffic stream may be considered once the lane 1 volume is known.

B.5.3.5 Level of Service Criteria

Level-of-service criteria for merge, v_m, diverge, v_d, and freeway, v_f, flow checkpoints are given in Table B5-12. The criteria for levels of service are similar to those given in B.3.2.1, but define operational conditions of merging and diverging.

Note that criteria are stated in terms of flows. As in B5-1 and B5-2, computational procedures include the conversion of peak-hour volumes to equivalent hourly flows representing flow during the peak 15-min interval.

The criteria of Table B5-11 are not specifically correlated to measures of operational quality. They are intended, however, to reflect flows which may be accommodated while permitting the freeway as a whole to operate at the designated level of service in the vicinity of the ramp. Thus, the quality of operations is expected to be as described in B5-1, with some local turbulence in lane 1.

Level-of-service A represents unrestricted operation. Merging and diverging vehicles have little effect on other freeway flows. Merging is smoothly accomplished with only minor speed adjustments required to occupy gaps; diverge movements encounter no significant turbulence.

At level-of-service B, merging vehicles have to adjust their speed slightly to occupy lane 1 gaps; diverging vehicles still do not experience any significant turbulence. Freeway vehicles not involved in merging or diverging movements are not seriously affected, and flow may be described generally as smooth and stable.

Level-of-service C, though still stable, approaches the range in which small changes in flow result in large changes in operating quality. Both lane 1 and on-ramp vehicles must adjust their speed to accomplish smooth merging, and under heavy on-ramp flows, minor ramp queuing may occur. Some slowing may also occur in diverge areas. Turbulence from on- and off-ramp manoeuvres is more widespread, and the effects of this turbulence may extend into freeway lanes adjacent to lane 1. Overall speed and density of freeway vehicles are not expected to deteriorate seriously.

At level-of-service D, smooth merging becomes difficult to achieve. Both entering and lane 1 vehicles must frequently adjust their speed to avoid conflicts in the merge area. Slowing in the vicinity of diverge areas is also significant. Turbulence from merge and diverge movements affects several freeway lanes. At heavily used on-ramps, ramp queues may become a disruptive factor.

Level-of-service E represents capacity operation. Merge movements create significant turbulence, but continue without noticeable freeway queuing. On-ramp queues, however, may be significant. Diverge movements are significantly slowed, and some queuing may occur in the diverge area. All vehicles are affected by turbulence, and vehicles not involved in ramp movements attempt to avoid this turbulence by moving towards the median lanes.

At level-of-service F all merging is on a stop-and-go basis, and ramp queues and lane 1 breakdowns are extensive. Much turbulence is created as vehicles attempt to change lanes to avoid merge and diverge areas. Considerable delay is encountered in the vicinity of the ramp terminal (and perhaps for some distance upstream on the freeway), and conditions may vary widely, from minute to minute, as unstable conditions create "waves" of alternatively good and forced flow.

B.5.3.6 Ramp Terminal Analysis

The procedure for analysis of ramp-freeway terminals is to find the level of service for known existing or forecast volumes. Design therefore is developed by trial-and-error analyses.

Following is a step-by-step procedure for the analysis of ramp terminals.

<u>Step 1 - Establish Ramp Geometry and Volumes</u>

The establishment of a configuration includes the type, location of, and volumes on, adjacent ramps. Configuration is also the basis for selection of a nomograph for computation of lane 1 volume. Because nomographs deal primarily with ramp pairs, an individual ramp with both upstream and downstream adjacent ramps will often be considered twice, as part of a pair with each. For initial consideration, any adjacent ramp within 1800 m of the subject ramp should be treated as influencing ramp terminal behaviour. Individual nomographs include more detailed criteria for when an "adjacent" ramp may be considered to be isolated, and when it must be considered as part of a combination with adjacent ramps.

It should be noted that all nomographs and accompanying equations are calibrated in terms of mixed vehicles per hour for a full hour (v/h). Thus, the lane 1 volume computation occurs before volumes are converted to equivalent flows in passenger cars per hour (pc/h).

Table B5-12
LEVEL-OF-SERVICE CRITERIA FOR CHECKPOINT FLOW AT RAMP-FREEWAY TERMINALS

Level of Service	Merge Flow[a] pc/h	Diverge Flow[b] pc/h	Freeway Flow pc/h/lane[c] Design Speed, km/h 120	110	100	90	80
A	≤600	≤650	760	680	d	d	d
B	≤1000	≤1050	1140	1060	1000	d	d
C	≤1450	≤1500	1600	1480	1400	1330	1270
D	≤1750	≤1800	1940	1780	1720	1640	1580
E	≤2000	≤2000	2000	2000	2000	1950	1900
F	Widely Variable						

a Lane-1 flow plus ramp flow for one-lane, right-side on-ramps.
b Lane-1 flow upstream of off-ramp for one-lane, right-side ramps.
c Freeway flow per lane upstream of off-ramp and/or downstream of on-ramp.
d LOS not attainable due to design speed.

Step 2 - Compute Lane 1 Volume

Lane 1 volume is computed using either one of 13 nomographs Figures B5-14 to B5-26, for 4-lane and 6-lane freeways. Tables B5-5 and B5-6 may be used where there is no available nomograph. Table B5-13 gives an index to these procedures. The choice of a specific nomograph depends on (1) the ramp configuration in conjunction with adjacent ramps, (2) the number of lanes on the freeway, and (3) whether the ramp in question is the first or second of a paired configuration.

Each of the nomographs contains a complete set of instructions for use, and details the conditions under which use is acceptable. These instructions and conditions should be carefully noted, particularly where an approximation is involved. Special instructions for such cases are provided. The equation for each nomograph is also prominently displayed. Where greater precision is desired, the direct use of the equation is recommended, although for many cases the precision provided by nomographs is adequate.

Step 3 - Convert all Volume to Passenger Cars Per Hour

All lane 1 volumes, ramp volumes, and freeway volumes must be converted to equivalent volumes in passenger cars per hour. Volumes in mixed vehicles per hour may be converted to passenger cars per hour by dividing by the appropriate heavy vehicle factor, f_{hv}, computed using procedures described in B5-1.

Before converting lane 1 volume to passenger cars per hour, it is necessary to determine truck presence in this lane. Figure B5-12 or local data are used to estimate the percentage of total freeway trucks in lane 1, from which the proportion of trucks in the lane 1 volume may be computed.

Step 4 - Compute Checkpoint Volumes

For each ramp analysis, there are up to three checkpoint volumes for each ramp or pair of ramps:

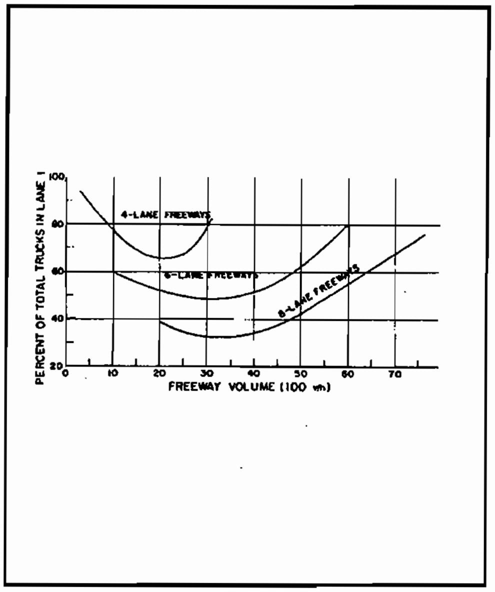


Figure B5-12
Truck Presence in Lane 1

1. Merge volume, V_m, - In any merge condition, two lanes merge to form a single lane. The merge volume is the sum of the volumes in the two lanes which join. In the most common case of a one-lane, right-side on-ramp, the merge volume equals the sum of the ramp volume plus the lane 1 volume immediately in advance of the ramp: $V_m = V_r + V_1$.

2. Diverge volume, V_d - The diverge volume is the total volume in a freeway lane immediately upstream of a point where the lane divides into two separate lanes. For the most common case of a one-lane, right-side, off-ramp, the diverge volume equals the lane 1 volume immediately in advance of the ramp: $V_d = V_1$.

3. Total freeway volume V_1 - The total volume on the freeway is checked at critical points. It is generally checked immediately downstream of an on-ramp.

Figure B5-13 illustrates the computation of checkpoint volumes for the case of an on-ramp followed by an off-ramp. Only one freeway volume checkpoint is needed, and that it is taken at a point between the two ramps where the freeway volume is at a maximum. This is consistent with the procedure outlined above, because the point selected is both upstream of the off-ramp and downstream of the on-ramp.

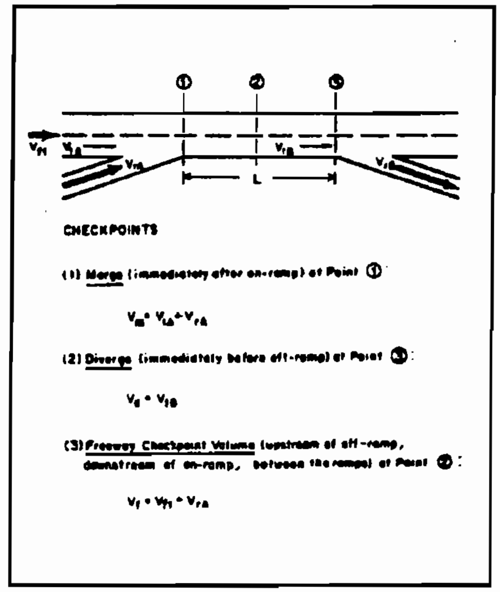


Figure B5-13
Computation of Checkpoint Volumes

Step 5 - Convert Checkpoint Volumes to Peak Flows

Before comparing checkpoint volumes with the level-of-service criteria of Table B5-11, they must be adjusted to reflect peak flows rather than full-hour volumes. This is accomplished by dividing each checkpoint volume by the peak-hour factor (PHF). Off-peak periods may be checked similarly.

Step 6 - Find Relevant Levels of Service

The level of service for a given analysis is found by comparing the checkpoint flows for merging, diverging, and total freeway flows with the criteria given in Table B5-12.

In many cases, the various operational elements (merges, diverges, freeway flows) will not have the same level of service. In such cases, the worst resultant LOS is assumed to govern the overall operation of the section in question. The analysis, however, will clearly identify those operational elements controlling the condition.These elements are then candidates for improvement if the resulting LOS is unacceptable. Thus, if a merge is a congesting element in a segment of freeway, efforts at improvement would be targeted at the design and operation of the troublesome merge point.

It is desirable to have point locations such as ramp terminals operating in balance with the freeways a whole. The most desirable operation would have the LOS of merge and diverge points equal to or better than the LOS for total freeway volume. Where merge and/or diverge points are the controlling element on a freeway segment, point congestion disrupts overall operation and prohibits the freeway from achieving a better level of service. Improvements at such locations should, therefore, be directed at removing point impediments and allowing the total freeway flow to determine operating conditions.

B.5.3.7 Ramp Roadways

There is very little information concerning operational characteristics on ramp roadways. Because most operational problems occur at ramp terminals, most quantitative studies have been concerned with terminal operations, rather than the ramp roadway itself.

Ramps differ considerably from the freeway mainline in that:

1. They are roadways of limited length and width (often one lane).
2. The design speed of the ramp is frequently lower than that of the roadways it connects.
3. On single-lane ramps, where passing is not possible, the adverse effect of trucks and other slow-moving vehicles is more pronounced than on a multilane roadway.
4. Acceleration and deceleration often take place on the ramp itself.
5. At ramp-street system interfaces, queuing may develop on the ramp.

Because of these distinct characteristics, it is difficult to adjust basic freeway criteria to approximate criteria for ramps. Service flow rates for levels of service are not as easily found, nor are there clear definitions of what type of operation is associated with each level. Table B5-14 gives approximate service flows for ramp roadway. Capacity estimates and flows were taken at similar v/c ratios as for the various levels of service on freeways. Available data do not permit each level to be precisely described in terms of operating characteristics.

These values may be adjusted for heavy vehicle presence and lane width restrictions using the factors of B.5.1. Their use in this context is, however, approximate.

Table B5-14 refers only to the ramp roadway itself. Even though up to 1,700 pc/h may be accommodated in a single-lane ramp, this does not guarantee that they can be accommodated in a single-lane ramp terminal, or at the ramp-street terminal. As a general rule-of-thumb, where flows exceed 1500 pc/h, a two-lane ramp is required.

Further, even where a one-lane ramp and ramp terminal are sufficient from the capacity point of view, a two-lane ramp is generally provided if:

1. The ramp is longer than 300 m to provide opportunities to pass stalled or slow-moving vehicles.
2. Queues are expected to form on the ramp from a controlled ramp-street terminal, to provide additional storage.
3. The ramp is located on a steep grade or has minimal geometrics.

If a two-lane ramp is provided for any of the above reasons, it is generally tapered to a single lane at the ramp-freeway terminal.

Drivers have difficulty maintaining two-lane flow on loop ramps because of their severe horizontal alignment. In cases where two-lane loop ramps are deemed necessary, lanes must be wider than 3.75 m. Lane-widening on loop ramps is based on the off-tracking characteristics of trucks.

These guidelines are useful in design where alternative ramp configurations may be developed for detailed analysis using ramp-freeway terminal procedures. In analysis, the total ramp flow may be checked to ensure that adequate capacity is provided. Rarely, however, will the ramp roadway itself be a controlling factor in either design or analysis.

B.5.4 Climbing Lanes (refer to Highway Capacity Manual, 1985)

Table B5-13

INDEX TO NOMOGRAPHS FOR LANE-1 VOLUMES

CONFIGURATION	4-LANE FREEWAY (2 LANES EACH DIRECTION)		6-LANE FREEWAY (3 LANES EACH DIRECTION)		8-LANE FREEWAY (4 LANES EACH DIRECTION)	
	1st RAMP	2nd RAMP	1st RAMP	2nd RAMP	1st RAMP	2nd RAMP
Isolated, One Lane On-Ramp	Fig. B5-14	—	Fig. B5-19	—	Fig. B5-22	—
Isolated, One Lane Off-Ramp	Fig. B5-15	—	Fig. B5-20	—	Approximate Using Table B5-15 and Fig. B5-16	—
Adjacent One-Lane On-Ramps	Fig. B5-14	Fig. B5-18	Fig. B5-19	Fig. B5-21	Approximate Using Table B5-15 and Fig. B5-16	Approximate Using Table B5-15 and Fig. B5-16
Adjacent One-Lane Off-Ramps	See Note 1	Fig. B5-15	See Note 2	Fig. B5-20	Approximate Using Table B5-15 and Fig. B5-16	Approximate Using Table B5-15 and Fig. B5-16
On-Ramp Followed by Off-Ramp	Fig. B5-14	Fig. B5-16	Fig. B5-19	Fig. B5-20	Fig. B5-23	Approximate Using Table B5-15 and Fig. B5-16
Off-Ramp Followed by On-Ramp	Treat as Isolated Ramps			Fig. B5-19	Treat as Isolated Ramps	
Loop Ramps	Fig. B5-17	Fig. B5-16	Fig. B5-19	Fig. B5-20	Fig. B5-23	Approximate Using Table B5-15 and Fig. B5-16
Two-Lane On-Ramps	See Note 3	—	Fig. B5-24	—	See Note 3	—
Two-Lane Off-Ramps	See Note 4	—	Fig. B5-25	—	See Note 4	—
Addition of Lane at On-Ramp	Merge criteria in Table B5-12 may be applied directly to the on-ramp flow rate as a checkpoint.					
Dropping a Lane to the Off- at Off-Ramp	Diverge criteria in Table B5-12 may be applied directly to the off-ramp flow rate as a checkpoint.					
Major Junctions (I, A, B)	Assume that line B carries an amount of traffic equal to the merge checkpoint volume in table B5-12 for the assumed level of service. Ramp lane A then carries the remaining ramp traffic. Compute lane 1 volume using Fig. B5-14 (4-lane freeway), Fig. B5-18 (6 lane freeway), Fig. B5-22 (8 lane freeway), entering with ramp volume = lane A volume. Find checkpoint levels of service. Continue computations until assumed LOS agrees with results.					
Major Diverges	Not Available	—	Fig. I.5-13	—	Not Available	—

Table B5-14
SERVICE FLOWS FOR
SINGLE-LANE[b] RAMPS

Level of Service	Service Flow pc/h Ramp Design Speed km/h <40	40-50	50-60	60-70	70-80	>80
A	a	a	a	a	a	600
B	a	a	a	900	900	900
C	a	a	1150	1250	1250	1300
D	a	1350	1450	1550	1550	1600
E	1300	1600	1650	1650	1650	1700
F	Widely Variable					

a Level of Service is not attainable due to restricted design speed
b For two-lane ramps multiply values in the table by:

for <40 km/h,	1.7
for 40-50 km/h,	1.8
for 50-60 km/h,	1.9
for >60 km/h,	2.0

Table B5-15
THROUGH TRAFFIC[a] REMAINING IN LANE 1

Total Through Volume in one Direction v/h	Through Traffic Volume Remaining in Lane 1, % (approximate)		
	8-lane Freeway	6-lane Freeway	4-lane Freeway
≥6500	10	-	-
6000 - 6490	10	-	-
5500 - 5999	10	-	-
5000 - 5499	9	-	-
4500 - 4999	9	18	-
4000 - 4499	8	14	-
3500 - 3999	8	10	-
3000 - 3499	8	6	40
2500 - 2999	8	6	35
2000 - 2499	8	6	30
1500 - 1999	8	6	25
≤1499	8	6	20

a Through traffic not involved in any ramp within 1200 m of the subject location.

Table B5-16

RAMP TRAFFIC IN LANE 1

Distance From Bullnose, m	Ramp Traffic in Lane 1, % On-Ramp	Ramp Traffic in Lane 1, % Off-Ramp
0	100	100
200	96	99
300	60	95
400	37	85
500	27	75
600	20	64
700	16	53
800	13	40
900	10	30
1000	10	20
1200	10	10

NOTE: Use the percentage in this table only if it is greater that than given by Table B5-15.

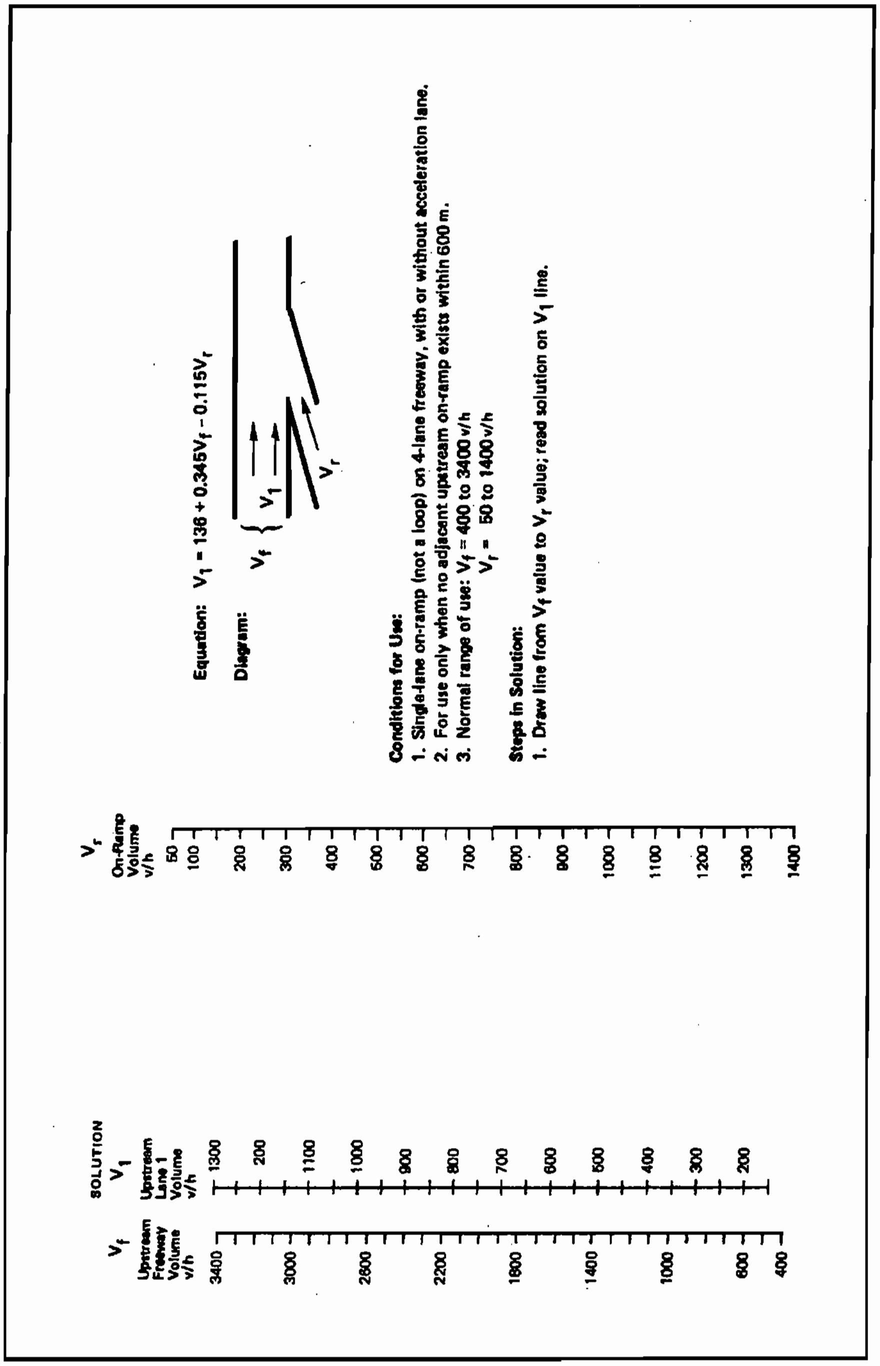


Figure B5-14
Determination of Lane-1 Volume Upstream of One-Lane On-Ramps on Four-Lane Freeways (Two Lanes in Each Direction)

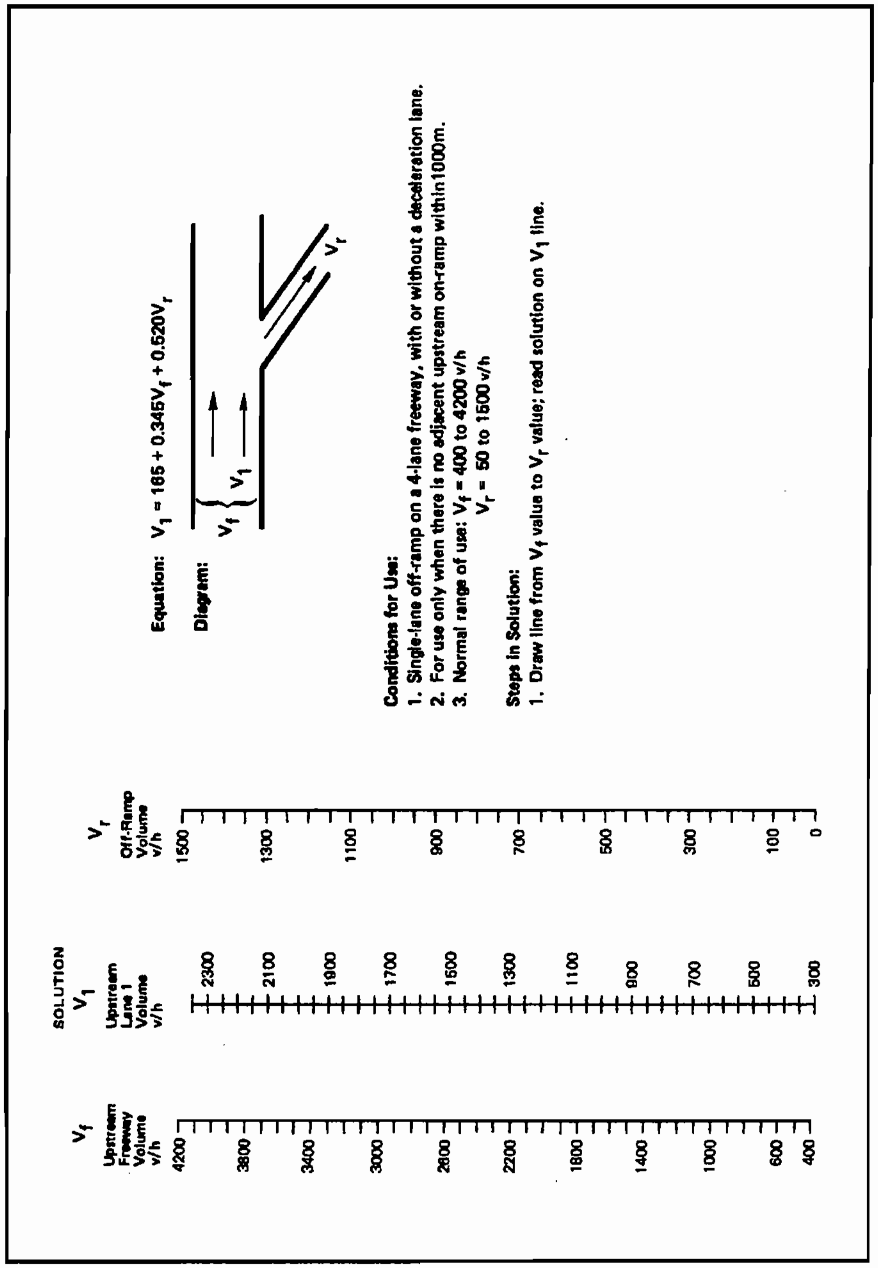


Figure B5-15
Determination of Lane-1 Volume
Upstream of One-Lane Off-Ramps on Four-Lane Freeways
(Two Lanes in Each Direction)

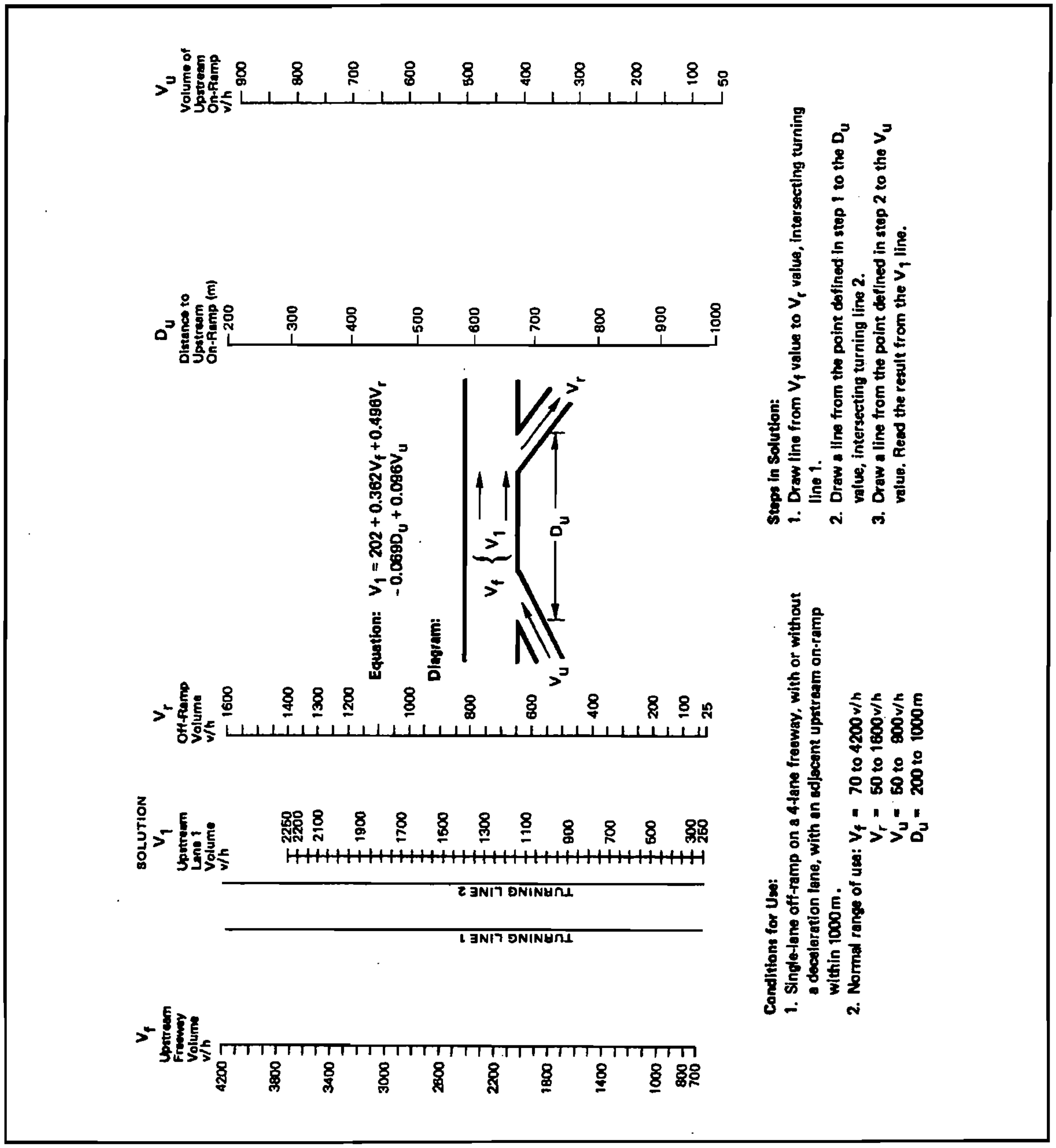


Figure B5-16

Determination of Lane-1 Volume
Upstream of One-Lane Off-Ramps on Four-Lane Freeways
(Two Lanes in Each Direction) with Adjacent Upstream On-Ramps

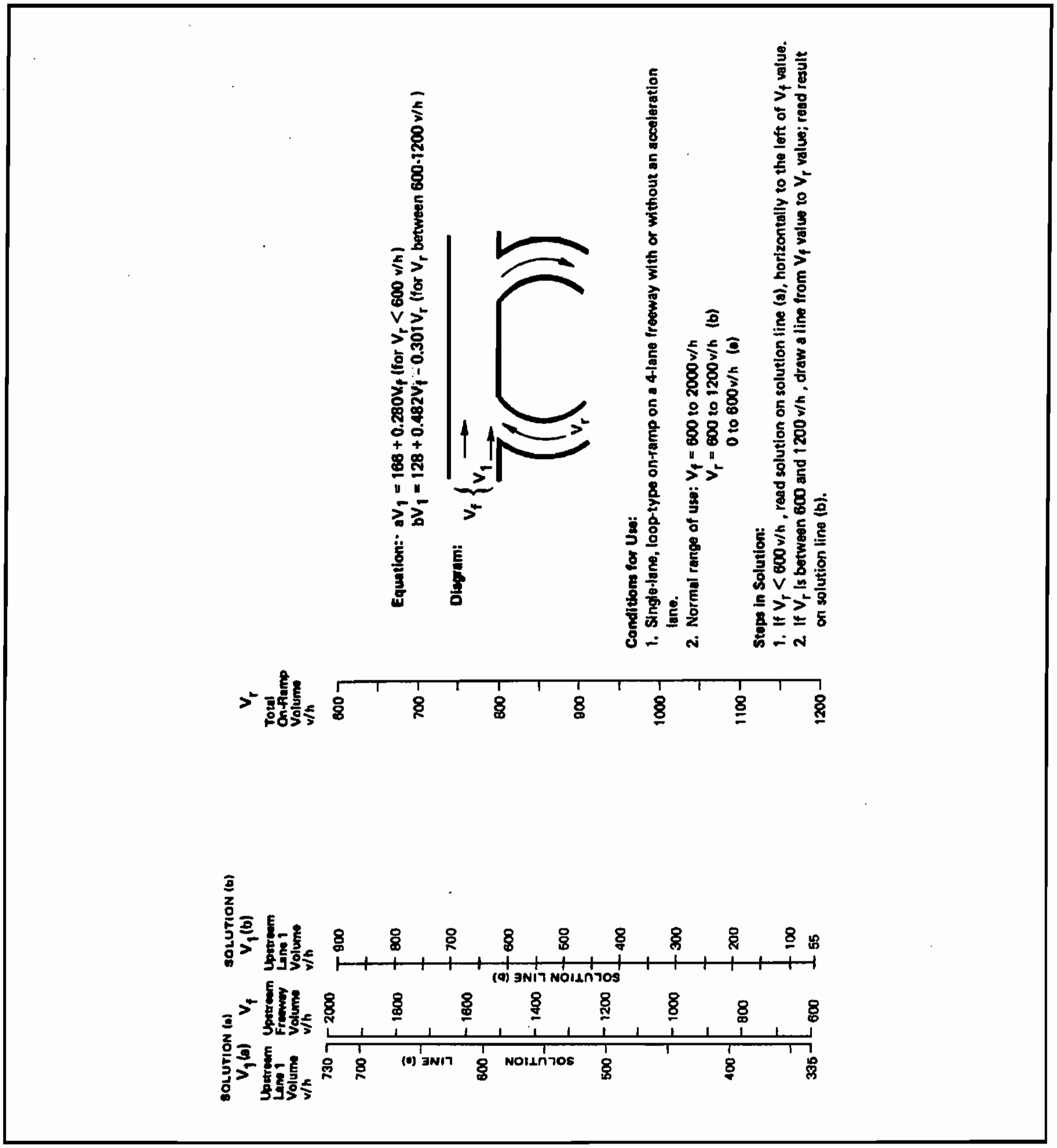


Figure B5-17
Determination of Lane-1 Volume
Upstream of One-Lane Loop-Type On-Ramps on Four-Lane Freeways
(Two Lanes in Each Direction)

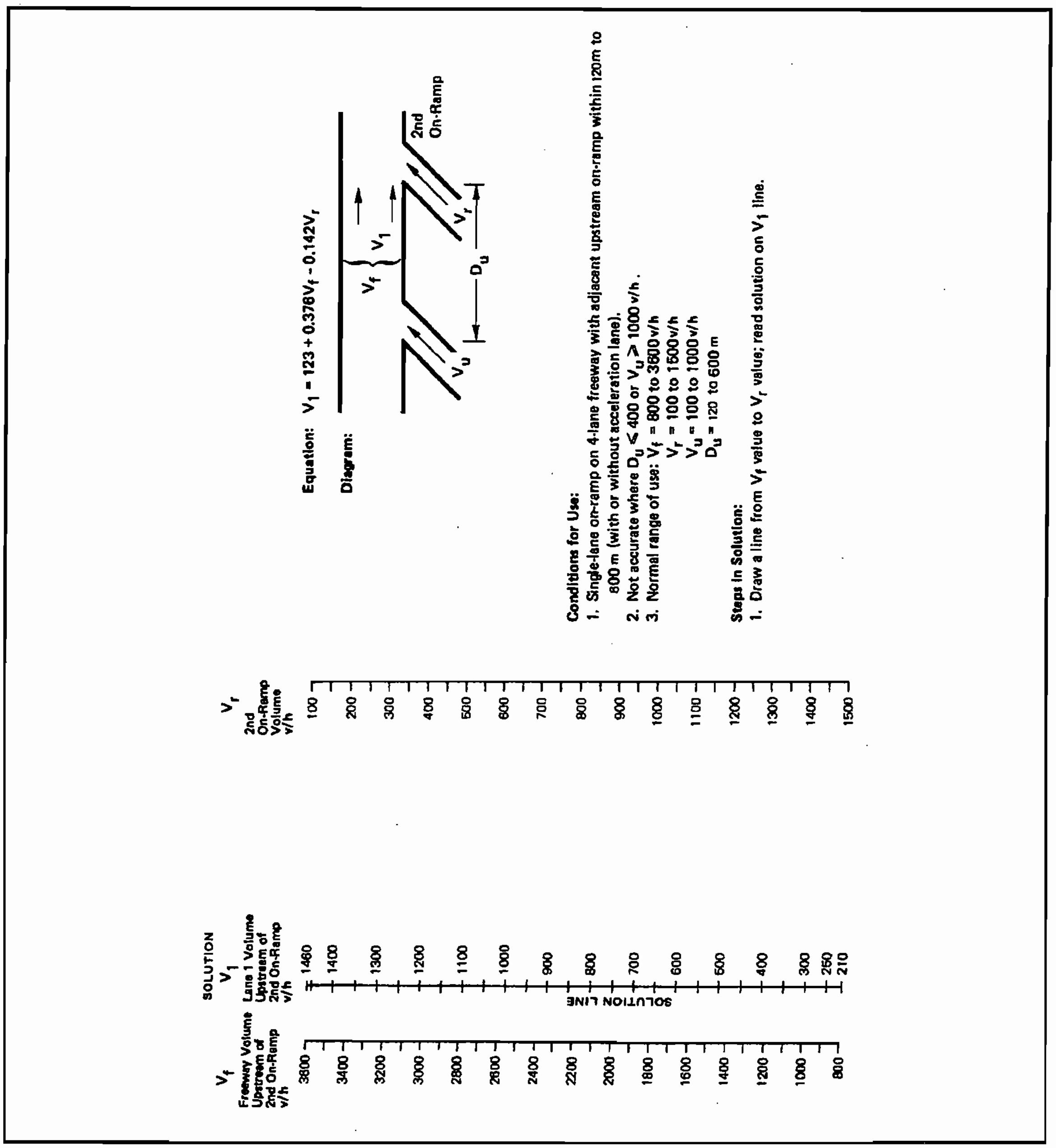


Figure B5-18
Determination of Lane-1 Volume Upstream of One-Lane On-Ramps on Four-Lane Freeways (Two Lanes in Each Direction) with Adjacent Upstream On-Ramps

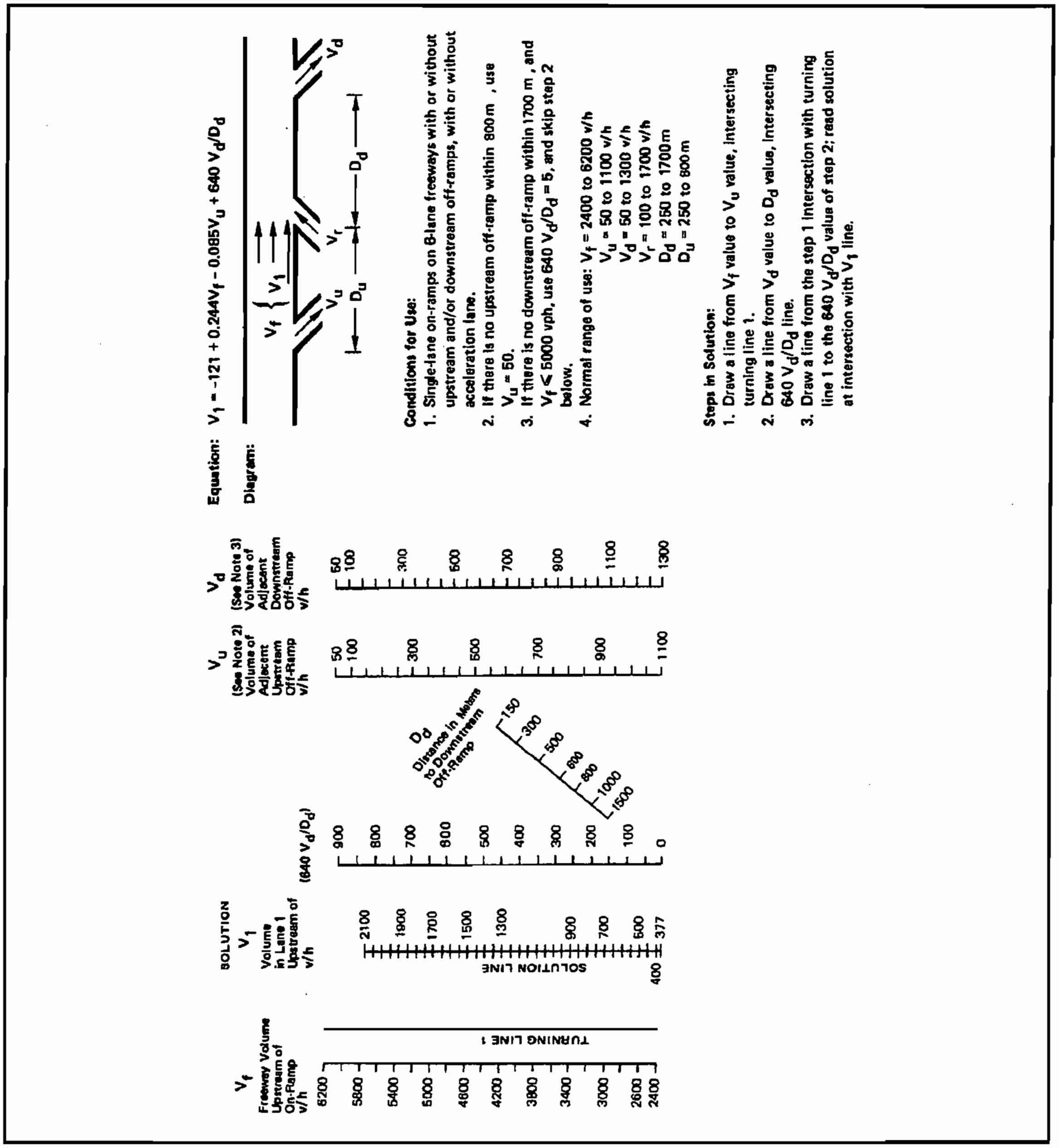


Figure B5-19
Determination of Lane-1 Volume Upstream of
One-Lane On-Ramps on Six-Lane Freeways
(Three in Each Direction)
With or Without Adjacent Off-Ramps

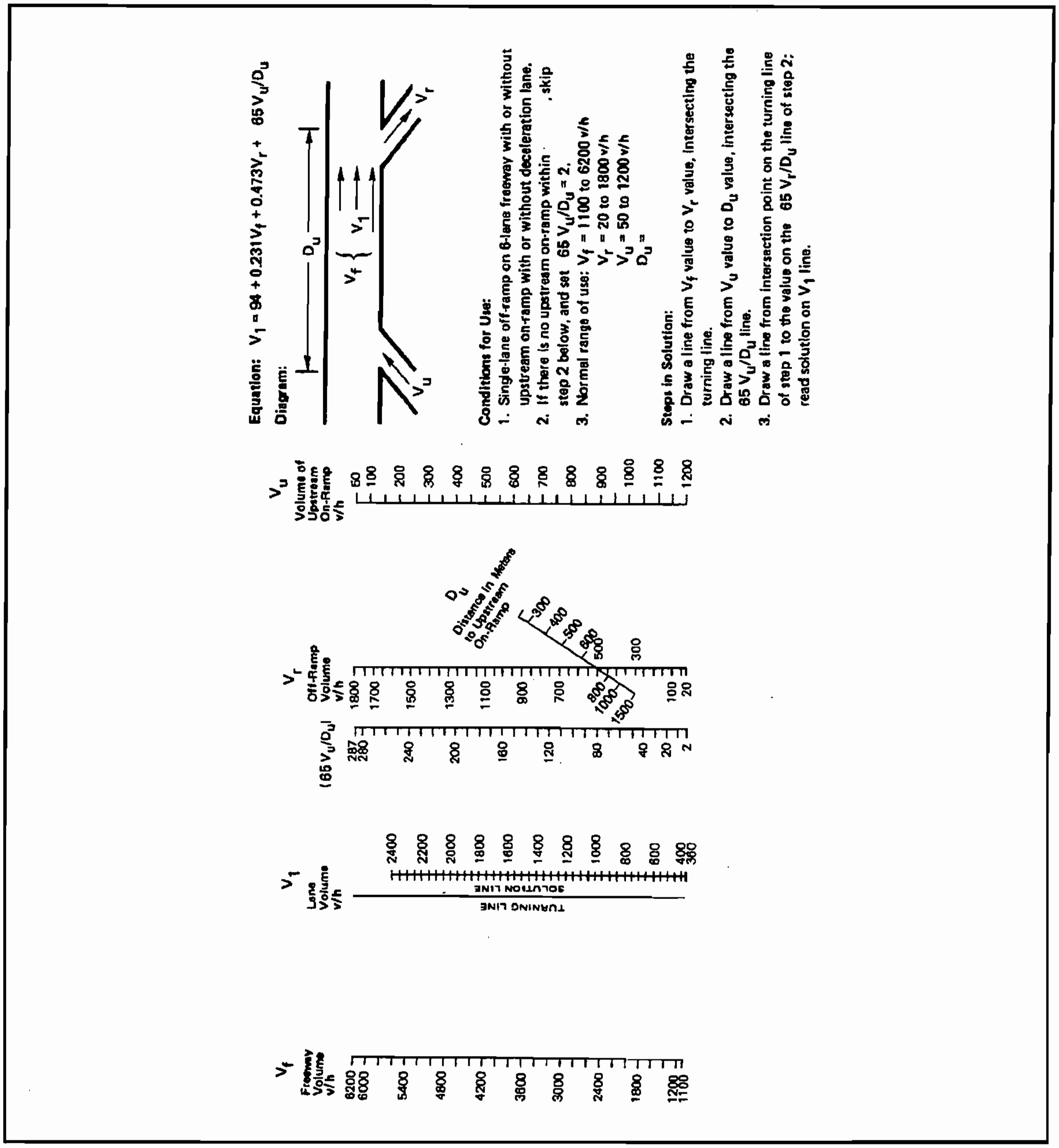


Figure B5-20
Determination of Lane-1 Volume Upstream Of One-Lane Off-Ramps on Six-Lane Freeways (Three Lanes in Each Direction)

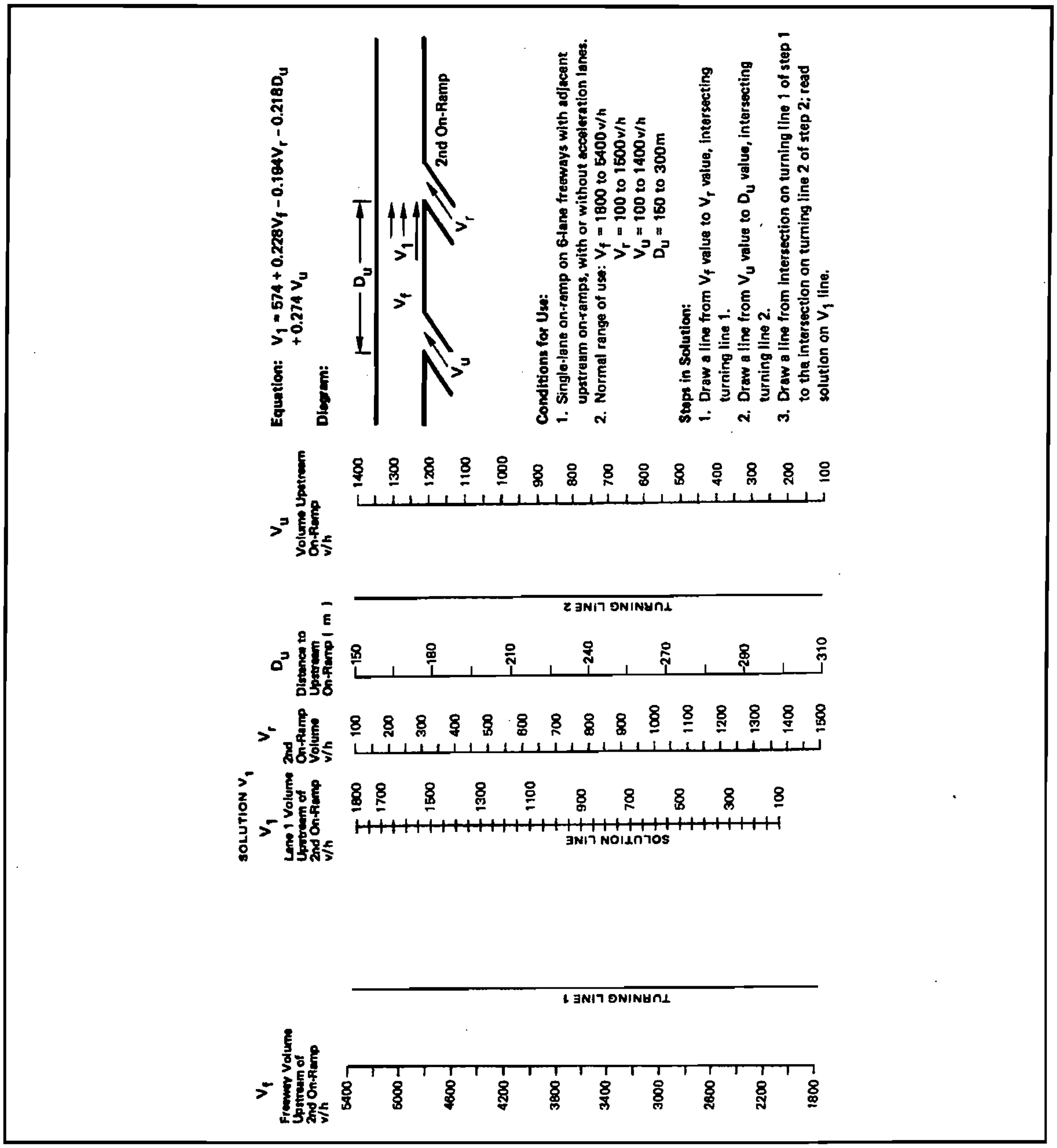


Figure B5-21
Determination of Lane-1 Volume Upstream of One-Lane On-Ramps on Six-Lane Freeways (Three Lanes in Each Direction) With Upstream On-Ramps

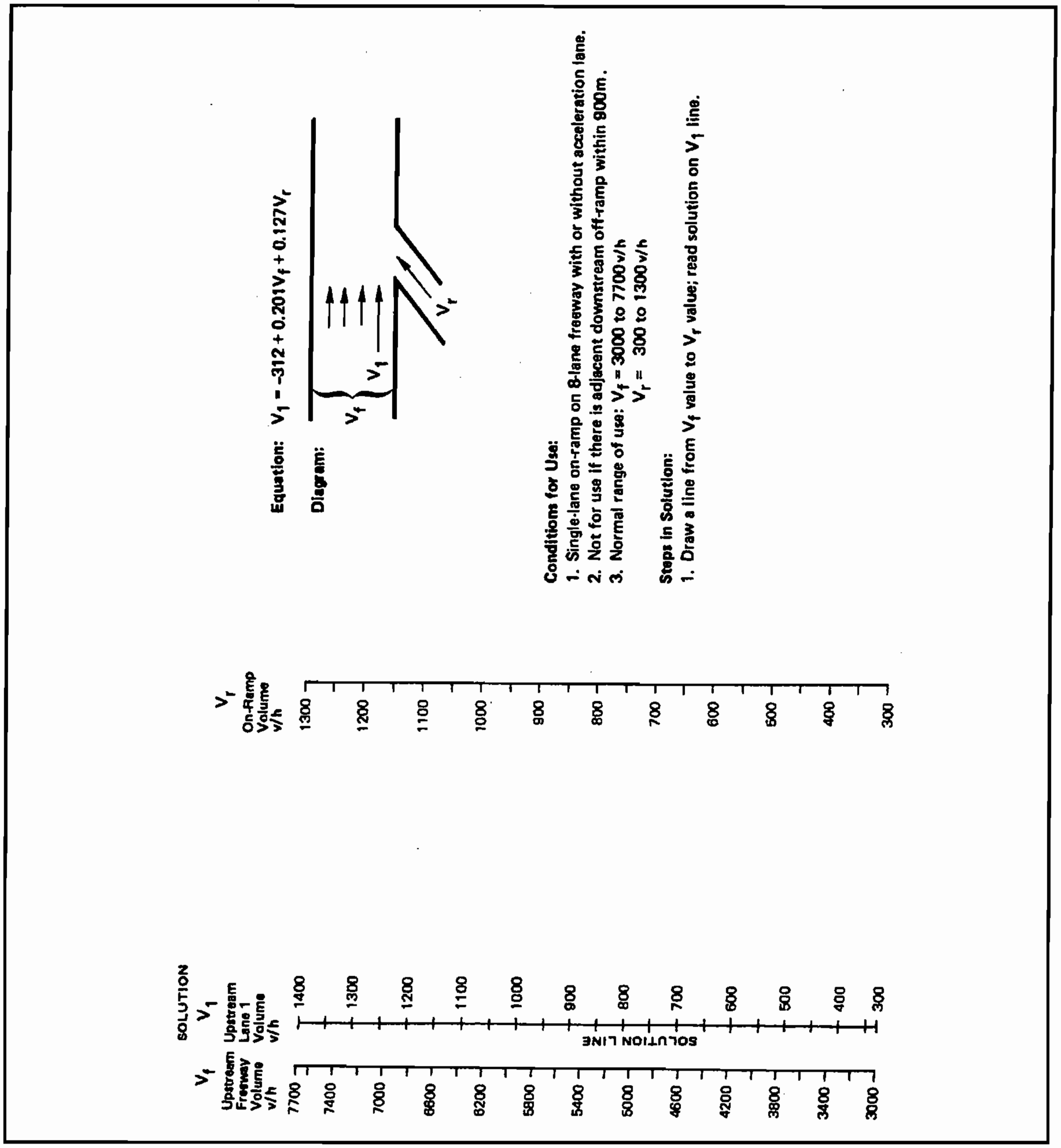


Figure B5-22
Determination of Lane-1 Volume Upstream Of One-Lane On-Ramps on Eight-Lane Freeways (Four Lanes in Each Direction)

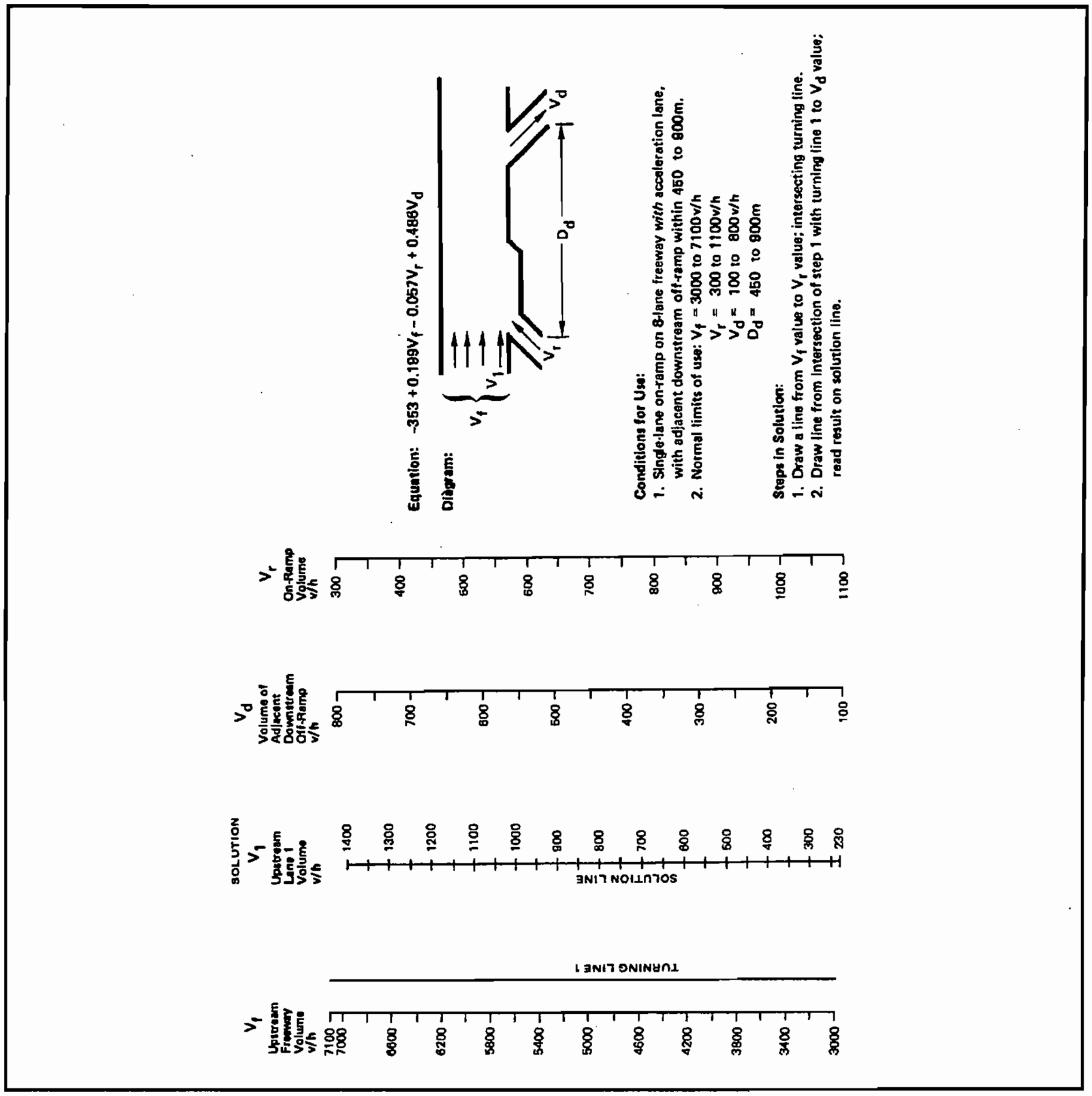
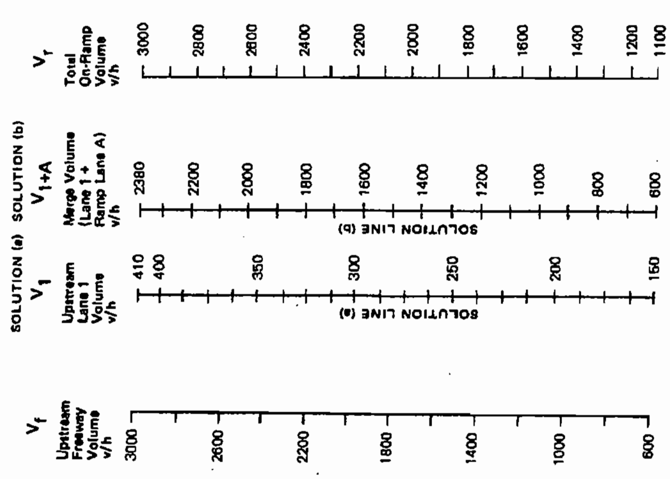
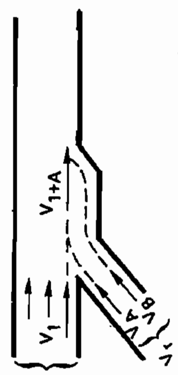


Figure B5-23
Determination of Lane-1 Volume Upstream of On-Ramps on Eight-lane Freeways (Four Lanes in Each Direction) With Adjacent Downstream Off-Ramps



Equation: (a) $V_1 = 54 + 0.070V_f + 0.049V_r$
(b) $V_{1+A} = -205 + 0.287V_f + 0.575V_r$

Diagram:



Conditions for Use:
1. Two-lane on-ramps on 6-lane freeways with acceleration lane of at least 240m in length.
2. Normal range of use: $V_f =$ 600 to 3000 v/h
$V_r =$ 1100 to 3000 v/h

Steps in Solution:
1. Draw line from V_f value to V_r value. Read V_1 on V_1 line, V_{1+A} on V_{1+A} line.
2. Compute $V_A = V_{1+A} - V_1$; $V_B = V_r - V_A$.
3. Check Level of Service for two merge points: $V_{m1} = V_1 + V_A$ and $V_{m2} = V_{1+A} + V_B$.

Figure B5-24
Determination of Lane-1 Volume Upstream of Two-Lane On-Ramps on Six-Lane Freeways (Three Lanes in Each Direction)

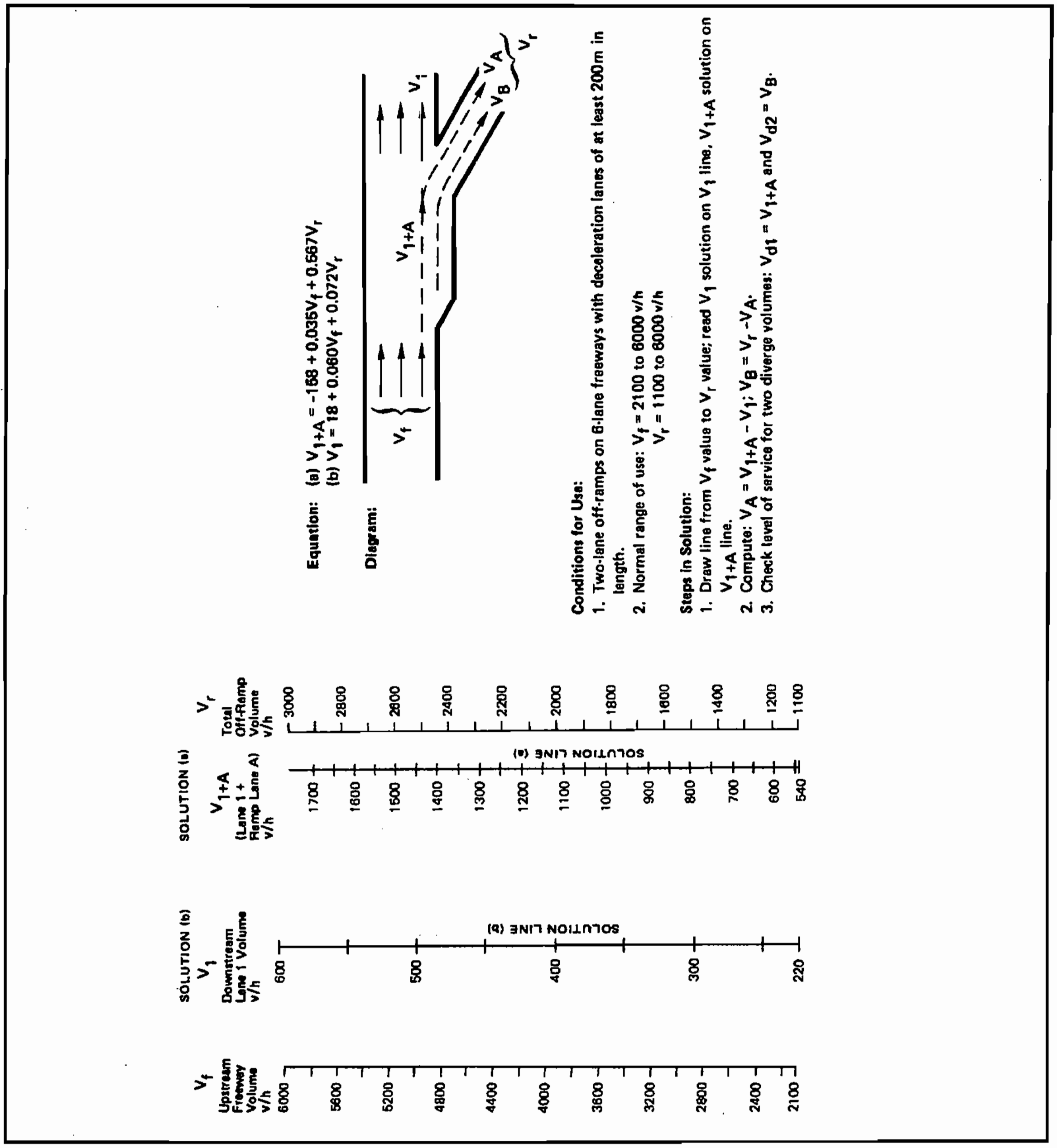


Figure B5-25
Determination of Lane-1 Volume Upstream of Two-Lane Off-Ramps on Six-Lane Freeways (Three Lanes in Each Direction)

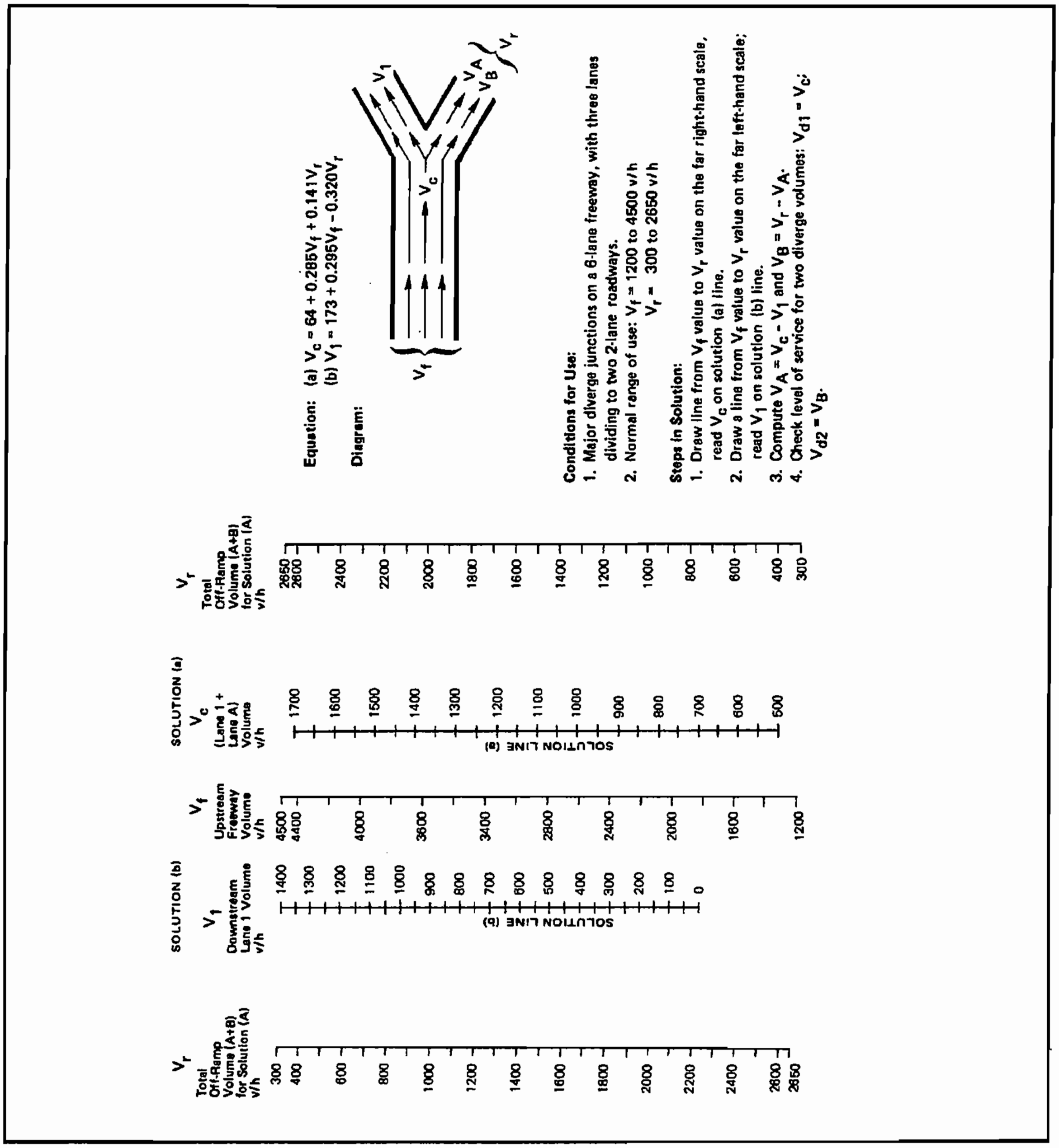


Figure B5-26
Determination of Critical Lane Volumes at a Major Fork
On a Six-Lane Freeway (Three Lanes in Each
Direction) Which Divides Into Two
Four-Lane Freeways (Two Lanes in Each Direction)

B.6 UNSIGNALIZED INTERSECTIONS

B.6.1 Conceptual Approach

Unsignalized intersections make up the vast majority of at-grade junctions in any street system. STOP and YIELD signs are used to assign the right-of-way to one street at such intersections. This designation forces drivers on the controlled street to judgmentally select gaps in the major street flow through which to execute crossing or turning manoeuvres. Thus, the capacity of the controlled legs is based on two factors:

- The distribution of gaps in the major street traffic stream.
- Driver judgment in selecting gaps through which to execute their desired manoeuvres.

Computational procedures depend on both factors: gap distributions in conflicting traffic streams and the gap acceptance behaviour of drivers at such intersections.

It is assumed that gaps in conflicting streams are randomly distributed. For this reason, the procedure is less reliable in situations in which conflicting flows are strongly platooned, such as the case at many urban intersections where the major street is part of a signalized network.

The impact of progression on the gap distribution in a major traffic stream can vary substantially. On one-way arterials, there are periodic large gaps between platoons through which minor street traffic may easily execute crossing and/or turning movements. Such a condition is likely to permit higher side-street capacities and better operations than the random arrivals assumed by the methodology of this chapter.

On two-way arterials, side street traffic may face a wide range of conditions. Platoons arrive in two directions on the major street. They may arrive such that considerable gaps exist between platoons, or they may arrive in a staggeredfashion (first from one direction, then the other). In the former case, side street crossings are easier to make than in the latter case, where the crossing vehicle is faced with a virtually endless platoon.

The method generally assumes that major street traffic is not affected by minor street flows. This assumption is generally good for periods when the operation is smooth and uncongested. When congestion occurs, it is likely that major flows will experience some impedance due to minor street traffic. Left turns from the major street are assumed to be affected by the opposing major street flow, and minor street traffic is affected by all conflicting movements.

The methodology also adjusts for the additional impedance of minor street flows on each other, and accounts for the shared use of lanes by two or three minor street movements, for example, right-turn, through, and left-turn movements sharing a single minor street lane.

To properly account for mutual impedances, the method is based on a prioritized regime of gap utilization. Gaps in the major street traffic flow are used by a number of competing flows. A gap used by a vehicle from one of these flows is no longer available for use by another vehicle. Gaps are utilized by vehicles in the following priority order:

1. Right turns from the minor street.
2. Left turns from the major street.
3. Through movements from the minor street.
4. Left turns from the minor street.

For example, if a left-turning vehicle on the major street and a through vehicle from the minor street are waiting to cross the major traffic stream, the first available gap (of acceptable size) will be taken by the left-turning vehicle. The minor street through vehicle must wait for the second available gap. In aggregate terms, a large number of such left-turning vehicles could use up so many of the available gaps that minor street through vehicles are severely impeded or unable to make safe crossing movements.

Right-turning vehicles from the minor street are not assumed to "use up" available gaps. Because such vehicles merely merge into gaps in the right-hand lane of the stream into which they turn, they require only a gap in that lane, not in the entire major street traffic flow. Further, a gap in the overall major street traffic could be simultaneously used by another vehicle. For this reason, the method does not assume that right turns from the minor street impede any of the other flows using major street gaps.

The basic structure of the procedure is as follows:

1. Define existing geometric and volume conditions for the intersection under study.

2. Determine the "conflicting traffic" through which each minor street movement, and the major street left turn, must cross.

3. Determine the size of the gap in the conflicting traffic stream needed by vehicles in each movement crossing a conflicting traffic stream.

4. Determine the capacity of the gaps in the major traffic stream to accommodate each of the subject movements that will utilize these gaps.
   - compute ideal capacity
   - adjust for impedance (some gaps are used by vehicles with a higher priority)

5. Adjust the capacities so found to account for the use of shared lanes.

These basic analysis steps are discussed in detail in the sections that follow.

B.6.2 Methodology

B.6.2.1 Data

Detailed descriptions of the geometrics, control, and volumes at the intersection are needed.Key geometric factors include:

1. Number and use of lanes.

2. Channelization.

3. Percent grade.

4. Curb radii and approach angle.

5. Sight distances.

The number and use of lanes is a critical factor. Vehicles in adjacent lanes can use the same gap in the traffic stream simultaneously (unless impeded by a conflicting user of the gap). When movements share lanes, only one vehicle from those movements may use each gap. Channelization is also important because it can be used to reduce impedance by separating conflicting flows from each other.

Volumes must be specified by movement. In general, full hour volumes are used in the analysis of unsignalized intersections because short-term fluctuations will generally not present major difficulties at such locations, however, flow may be used. Volume for movement i is designated as V_i. In cases where flows are used, the notation remains, but refers to the flow instead of volume.

By convention, subscripts 1 to 6 are used to define movements on the major street, and subscripts 7 to 12 to define movements on the minor street (as used in Figure B6-3 for example). Conversion of vehicles per hour to passenger cars per hour is accomplished using the passenger-car equivalent values given in Table B6-1. Note that the table accounts for both grade and vehicle type, and that even passenger cars must be adjusted if the intersection approach is on a grade.

In addition to the geometric and volume data noted above, it is necessary to record the average running speed of vehicles on the major roadway.

B.6.2.2 Conflicting Traffic Movements

The nature of conflicting movements at an unsignalized intersection is relatively complex. Each subject movement faces a different set of conflicts that are directly related to the nature of the movement. These conflicts are depicted in Figure B6-1, which illustrates the computation of the parameter:

V_{ci} = the "conflicting volume" for movement i, that is, the total volume which conflicts with movement i, expressed in vehicles per hour.

Table B6-1

PASSENGER-CAR EQUIVALENTS FOR UNSIGNALIZED INTERSECTIONS

TYPE OF VEHICLE	GRADE				
	-4%	-2%	0%	+2%	+4%
Motorrcycles	0.3	0.4	0.5	0.6	0.7
Passenger Cars	0.8	0.9	1.0	1.2	1.4
SU/RV's[a]	1.0	1.2	1.5	2.0	3.0
Combination Veh.	1.2	1.5	2.0	3.0	6.0
All Vehicles[b]	0.9	1.0	1.1	1.4	1.7

[a] Single-unit trucks and recreational vehicles.
[b] If vehicle composition is unknown, these values may be used as an approximation.

The right-turn movement from the minor street, for example, is in conflict with only the major street through movement in the right-hand lane into which right-turners will merge. Figure B6-1 includes one-half of the right-turn movement from the major street, because this flow has been found to have a somewhat inhibiting effect on the subject movement. This may be caused by such vehicles approaching without using their turn indicator, causing the driver of a waiting vehicle to believe it will travel straight through the intersection and/or side frictions created as they turn into a lane adjacent to waiting vehicles.

Left turns from the major street are in absolute conflict with the total opposing through and right-turn flows, because they must cross the through flow and merge with the right-turn flow. The method does not differentiate between crossing and merging conflicts. Left turns from the major street and the opposing right turns from the major street are considered to merge, regardless of the number of lanes provided in the exit roadway.

Minor street through movements have a direct crossing or merging conflict with all movements on the major street, as indicated in Figure B6-1, except the right turn into the subject approach. Only one-half of this movement is included in the computation, for the same reasons as discussed above.

The left turn from the minor street is the most difficult manoeuvre to execute from an unsignalized intersection, and it faces the most complex set of conflicting flows. Conflicting volumes include all major street flows, in addition to the opposing right turn and through movement on the minor roadway.

B.6.2.3 Critical Gap and Capacity

<u>Critical Gap Size</u>

The critical gap is defined as the median time headway between two successive vehicles in the major street traffic stream that is accepted by drivers in a subject movement that must cross and/or merge with the major street flow. It is denoted as T_c, and is expressed in seconds. The critical gap depends on a number of factors, including:

1. The type of manoeuvre being executed (more complex manoeuvres require longer gaps).
2. The type of minor street control (STOP or YIELD - longer for STOP than YIELD).
3. The average running speed on the major street (longer gap required for higher speed).
4. The number of lanes on the major street (longer gaps required for more lanes).
5. The geometrics and environmental conditions at the intersection.

Values of critical gap are selected from Table B6-2 in a two-part process:

1. The basic critical gap size is selected from the first half of the table for the type of movement, type of control, and major street speed at the subject location.
2. Adjustments and modifications to the basic critical gap size are selected from the second half of the table for a variety of conditions, subject to the limitations given in the footnotes.

The population factor is incorporated because field experience indicates that drivers familiar with more congested traffic environments tend to select smaller gaps. Judgement may be used in applying this adjustment to reflect local driving habits.

The impact of poor sight distance is a complex factor requiring judgement. A site examination may be required to decide on whether or not to apply a factor and if so what value to use. Such factors as accident experience, driver response and gap acceptance, traffic volumes, and measured sight distances should be considered. Where such field examinations are not possible,computations should be done using arange of values to examine the sensitivity to this factor.

Potential Capacity for a Movement

The potential capacity of a movement is denoted as c_{pi} (for movement i), and is defined as the "ideal" capacity for a specific subject movement, assuming the following conditions:

1. Traffic on the major roadway does not block the minor road.
2. Traffic from nearby intersections does not back up into the intersection under consideration.
3. A separate lane is provided for the exclusive use of each minor street movement under consideration.
4. No other movements impede the subject movement.

The potential capacity in passenger cars per hour is selected from Figure B6-2, and is based on the conflicting traffic volume, V_c, in vehicles per hour, and the critical gap, T_c, in seconds. The figure is entered on the horizontal axis with the value of V_c. A vertical line is drawn to the appropriate "critical gap" curve. A horizontal line is drawn from the intersection with the "critical gap" curve to the vertical axis, where the result is read, in passenger cars per hour.

Impedance Effects

It has been noted that vehicles utilize gaps at an unsignalized intersection in a prioritized manner. When traffic becomes congested in a high-priority movement, it can impede lower priority movements from utilizing gaps in the traffic stream, and reduce the potential capacity of the mvoement. It should be noted that major street flows, and that "impedance" affects only minor street vehicles.

Right turns from the minor street do not generally impede other traffic elements, except for opposing left turns from the minor street where both movements are merging into the same traffic stream. Given the priority of gap usage:

- Left turns from the major street impede both through movements and left turns from the minor street.
- Through movements from the minor street impede left turns from the minor street.

In general, the impact of impedance is addressed by multiplying the potential capacity of a movement, c_{pi}, by a series of impedance factors, P_j, for each impeding movement j. These computations are illustrated in Figure B6-2, and result in finding the movement capacity, c_{mi}, which is the adjusted capacity of the movement. The "movement capacity" still assumes that the movement has exclusive use of a separate lane.Impedance factors, P_j, are found from FigureB6-3. They are based solely on the percent of potential capacity of the impeding movement used by existing demand. Consider the following example. A left-turn movement from a minor street at a T-intersection is impeded by the left turn from the major street. The latter movement has a potential capacity of 500 pc/h and a demand of 200 pc/h. Thus, the major street left turn uses 200/500 = 0.40, or 40% of its available capacity. Figure B6-3 is entered with this value, and an impedance factor of 0.68 is read. The potential capacity for the minor street left turn must then be multiplied by 0.68 to account for the impedance of the major street left turn.

Essentially, the computation of potential capacity assumes that all movements have exclusive access to available gaps. The availability of these gaps to lower priority movements is reduced as they are utilized by higher priority movements. This reduction is computationally represented in the impedance factors.

B.6.2.4 Shared Lane Capacity

The procedure so far has assumed that each minor street movement has the exclusive use of a lane. This is often not the case, and frequently two or three movements share a single lane on the minor approach. When this occurs, vehicles from different movements do not have simultaneous access to gaps, nor can more than one vehicle from the sharing movements utilize the same gap.

Occasionally, an intersection with wide corner radii allow vehicles approaching in the same laneto stop side-by-side. This acts to reduce or eliminate the adverse impact of the shared lane. Where several movements share the same lane, and cannot stop side-by-side at the stop line of the intersection, the following equation is used to compute the capacity of the shared lane:

$$C_{SH} = \frac{v_l + v_t + v_r}{[v_1/c_{ml}] + [v_t/c_{mt}] + [v_r/c_{mr}]}$$

where:

c_{SH} = capacity of the shared lane, in pc/h

v_l = volume or flow of left-turn movement in shared lane, in pc/h

v_t = volume or flow of through movement in shared lane, in pc/h

v_r = volume or flow of right-turn movement in shared lane, in pc/h

c_{ml} = movement capacity of the left-turn movement in shared lane, in pc/h

c_{mt} = movement capacity of the through movement in shared lane, in pc/h;

and

c_{mr} = movement capacity of the right-turn movement in shared lane, in pc/h

Only those movements included in the shared lane are included in the equation. If the shared lane includes only right-turn and through movements, both numerator and denominator terms for left-turns are deleted in the equation.

Table B6-2

CRITICAL GAP CRITERIA FOR UNSIGNALIZED INTERSECTIONS

VEHICLE MANOEUVRE AND TYPE OF CONTROL	BASIC GAP FOR PASSENGER CARS, s			
	AVERAGE RUNNING SPEED, MAJOR ROAD			
	50 km/h		90 km/h	
	NUMBER OF LANES ON MAJOR ROAD			
	2	**4**	**2**	**4**
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5

ADJUSTMENTS AND MODIFICATIONS TO CRITICAL GAP, s

CONDITION	ADJUSTMENT
RT from Minor Street: Curb radius > 15 m or turn angle < 60°	- 0.5
RT from Minor Street: Acceleration lane provided	- 1.0
All movements: Population ≥ 250,000	- 0.5
Restricted sight distance[a]	up to + 1.0

Notes : Maximum total decrease in critical gap = 1.0 s
Maximum critical gap = 8.5 s
For values of average running speed between 50 and 90 km/h, interpolate

[a] This adjustment is made for the specific movement impacted by restricted sight distance.

Table B6-3

LEVEL-OF-SERVICE CRITERIA FOR UNSIGNALIZED INTERSECTIONS

RESERVE CAPACITY (PC/H)	LEVEL OF SERVICE	EXPECTED DELAY TO MINOR STREET TRAFFIC
≥400	A	Little or no delays
300 - 399	B	Short traffic delays
200 - 299	C	Average traffic delays
100 - 199	D	Long traffic delays
0 - 199	E	Very long traffic delays
[a]	F	[a]

[a] When demand volume exceeds the capacity of the lane, extreme delays will be encountered with queuing which may cause severe congestion affecting other traffic movements in the intersection. This condition usually warrants improvement to the intersection.

Table B6-4

CAPACITY OF FOUR-WAY STOP-CONTROLLED INTERSECTIONS BY DEMAND

DEMAND SPLIT	CAPACITY [a, b] (v/h)
50/50	1,900
55/45	1,800
60/40	1,700
65/35	1,600
70/30	1,500

[a] Total capacity, all legs
[b] Based on two-lane by two-lane

Table B6-5

CAPACITY OF FOUR-WAY STOP-CONTROLLED INTERSECTIONS BY WIDTH

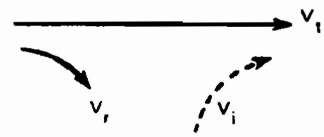
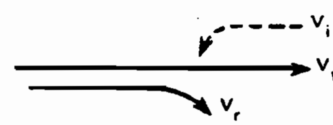
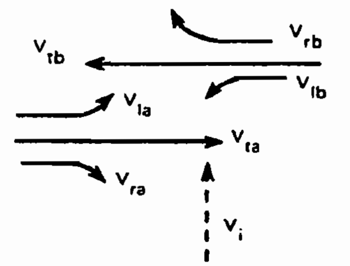
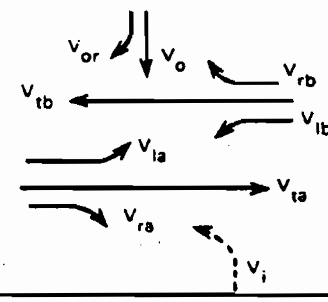
INTERSECTION TYPE	CAPACITY [a, b] (v/h)
2-lane by 2-lane	1,900
2-lane by 4-lane	2,800
4-lane by 4-lane	3,600

[a] Total capacity, all legs
[b] Based on 50/50 demand split

Table B6-6

LEVEL-OF-SERVICE C SERVICE VOLUMES FOR FOUR-WAY STOP-CONTROLLED INTERSECTIONS

	LOS C SERVICE VOLUME, V/H		
	NUMBER OF LANE		
DEMAND SPLIT	2 BY 2	2 BY 4	4 BY 4
50/50	1,200	1,800	2,200
55/45	1,140	1,720	2,070
60/40	1,080	1,660	1,970
65/35	1,010	1,630	1,880
70/30	960	1,610	1,820

Subject Movement	Conflicting Traffic, V_{ci}	Illustration
1. RIGHT TURN from minor street.	$1/2(V_r)^{**} + V_t^{*}$	
2. LEFT TURN from major street.	$V_r^{***} + V_t$	
3. THROUGH MVT from minor street.	$1/2(V_{ra})^{**\ ***} + V_{ta} + V_{la} + V_{rb} + V_{tb} + V_{lb}$	
4. LEFT TURN from minor street.	$1/2(V_{ra})^{**} + V_{ta} + V_{la} + V_{rb}^{***} + V_{tb} + V_{lb} + V_o + V_{or}$	

* V_t includes only the volume in the right hand lane.

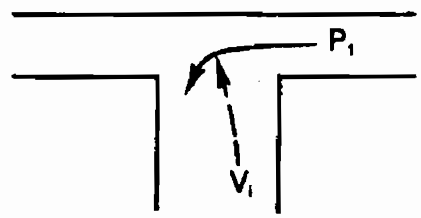
** Where a right-turn lane is provided on major street, eliminate V_r or V_{ra}.

***Where the right-turn radius into minor street is large and/or where these movements are STOP/YIELD-controlled, eliminate V_r (Case 2), and V_{ra} and/or V_{rb} (Case 4). V_{rb} may also be eliminated on multilane major streets.

Figure B6-1

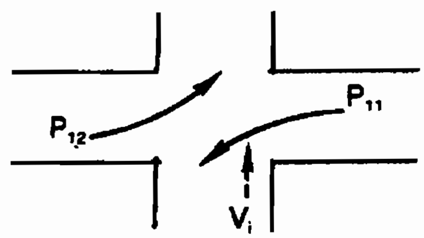
Conflicting Traffic Movements

1. *Left turns* from minor street at a T-intersection.



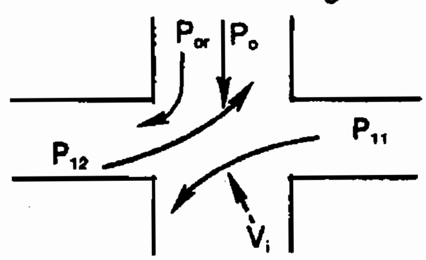
$$c_{mi} = c_{pi} \times P_1$$

2. *Through traffic* from minor street at a 4-leg intersection.



$$c_{mi} = c_{pi} \times P_{11} \times P_{12}$$

3. *Left turns* from minor street at a 4-leg intersection.



$$c_{mi} = c_{pi} \times P_{11} \times P_{12} \times P_o \times P_{or}$$

Figure B6-2

Illustration of Impedance Calculations

B.6.2.5 Level of Service Criteria

The calculations described above result in a solution for the capacity of each lane on the minor approaches to a STOP- or YIELD-controlled intersection. Level-of-service criteria for this methodology are stated in very general terms, and are related to general delay ranges. The criteria are given in Table B6-3 and are based on the reserve, or unused, capacity of the lane in question. This value is computed as:

$$c_R = c_{SH} - v$$

where:

c_R = reserve or unused capacity of the lane, in pc/h

c_{SH} = shared-lane capacity of the lane, in pc/h; and

v = total volume of flow rate using the lane, in pc/h

Caution should be used in the interpretation of these criteria. They are stated in general terms, without specific numeric values. It is, therefore, not possible to directly compare an unsignalized LOS with a signalized intersection analysis LOS in terms of specific delay values without collecting delay data directly at the subject site. The levels of service in this section are not associated with the delay values cited for signalized intersections in section B7.

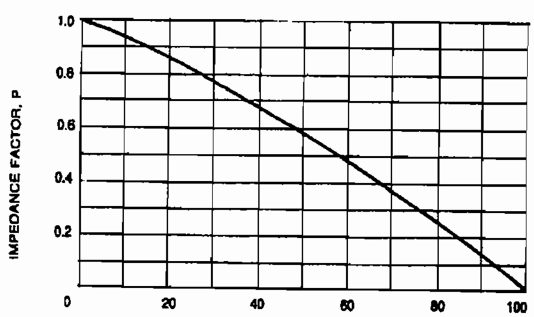


Figure B6-3

Impedance Factors as a Result of Congested Movements

B.6.3 Procedure

B.6.3.1 Data

Data required for calculations include:

1. Volumes by movement for the hour of interest.
2. Vehicle classification for the hour of interest.
3. Peak hour factor (if peak flows are being used as the basis for analysis).
4. Prevailing (average running) speed of traffic on the major street.
5. Number of lanes on the major street.
6. Number and use of lanes on the minor street approaches.
7. Grade of all approaches.
8. Other geometric features of interest: channelization, angle of intersection, sight distance, corner radii, acceleration lanes, etc.
9. Type of control on the minor approaches.

Because the methodology results in a qualitative evaluation of delay, it is also recommended, if possible, that some delay data be collected with the above information. This will allow for a better quantification and description of existing operating conditions at the location under study. It also allows for a more precise comparison with a signalized intersection analysis for which precise delay estimates are generated.

B.6.3.2 Sequence of Calculations

As the methodology is based on a prioritized use of gaps by vehicles at an unsignalized intersection, it is important that calculation be made in a particular order. The calculation sequence is the same as the priority of gap use, and movements are considered in the following order:

1. Right turns from the minor street.
2. Left turns from the major street.
3. Through movements from the minor street.
4. Left turns from the minor street.

To assist in maintaining the proper order of calculation worksheets are provided for the two principal types of intersections which are generally the subject of such analyses: four-leg intersections and T-intersections. The use of each of these in computational analysis is described in the sections below.

B.6.4 Analysis of 4-leg Intersections

Figure B6-3, B6-4 and B6-5 show worksheets for the analysis of 4-leg intersections.

Page 1 provides for the summary of demand volumes and basic geometric data. Page 2 is for the calculation of movement capacities for each movement. The equation are given on the worksheet and volumes are keyed to the diagrams on Page 1. Volumes are denoted by a (capital) V and flows by (small) v. Page 3 is usedto calculate shared-lane capacities, reserve capacities and level of service.

B.6.5 "T" Intersections

The analysis of "T" intersections follows the same steps as four-leg intersections, however, the procedure is very much simpler since there are fewer conflicting movements. Figure B6-6 provides a simplified worksheet.

The upper part of the sheet provides for summarizing the volume and geometric data, and for the adjustment of volumes, as for four-leg intersections. There are only six volumes to be considered, and only three of these need be converted to pc/h. If the intersection contains unusual geometric elements that are difficult to show on the worksheet, a schematic sketch should be developed and attached for clarity.

The middle part of the form is for the calculation of movement capacities. There are only three movements to be considered.

The lower part of the form provides for the calculation of shared-lane capacities. Because there are only two minor street flows, they either do or do not share a lane.

As was the case for four-leg intersections, it is often useful to calculate the reserve capacity of each movement as if each had a separate lane, even where a lane is shared. This will assist in the assessment of possible lane additions as a solution to a substandard operation.

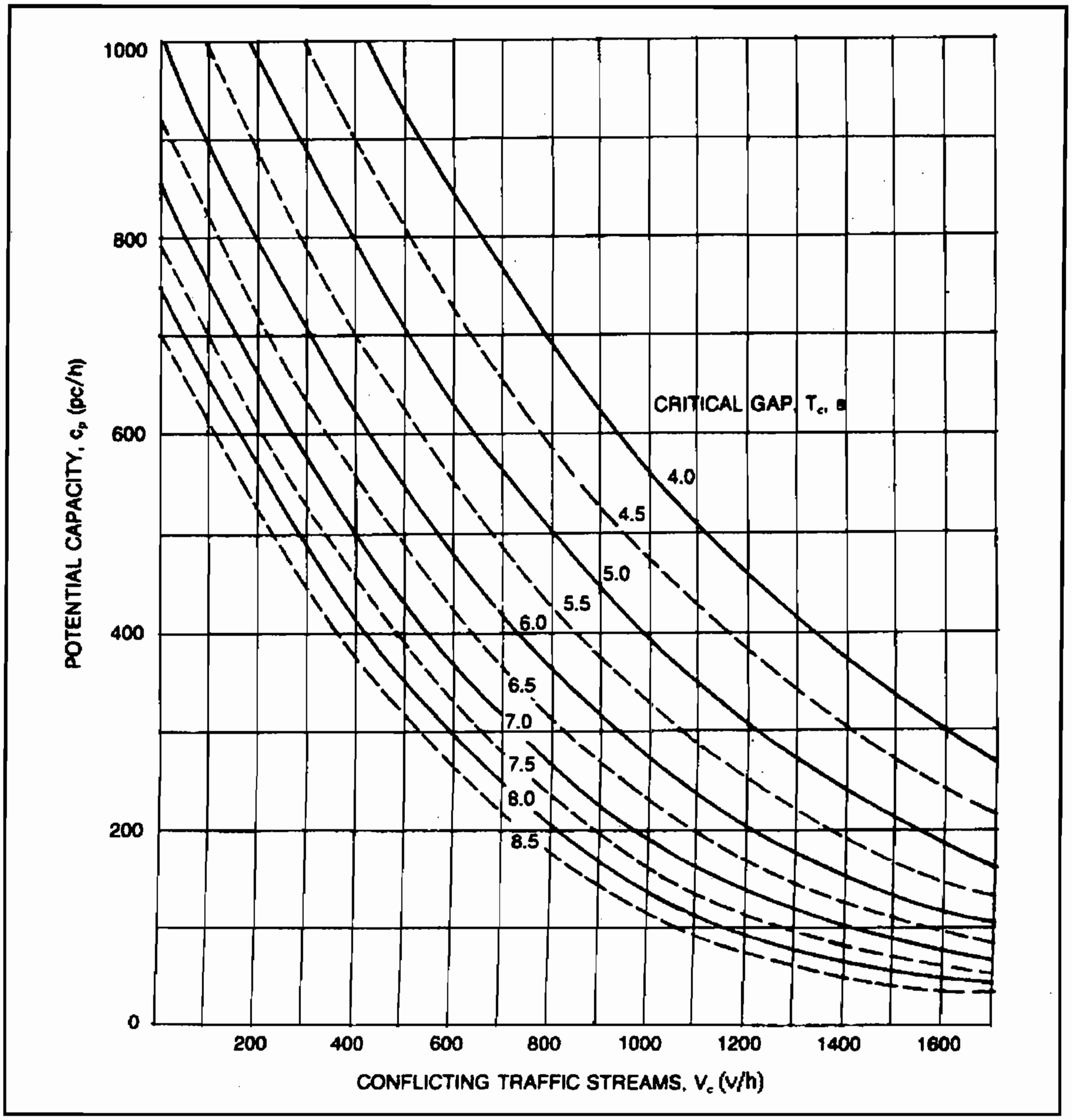


Figure B6-4

Potential Capacity of Unsignalized Intersections

B.6.6 Multiway Stop Control

The capacity of multiway STOP-controlled intersections is a function of the number of approach lanes, and of the departure headways of vehicles crossing from a stopped position. At capacity, operations are relatively predictable, with queues developing along each approach, and vehicles discharging in a regular rotational manner.

Table B6-4 gives typical capacity values for a two-lane by two-lane four-way STOP-controlled intersection. As the table indicates, capacity is greatest when demand volume is evenly split between the crossing facilities. Capacities as high as 1,900 v/h can be achieved at such intersections. A characteristic of intersections with a 50/50 demand is that vehicle delay tends to be uniform, and, because of the regular discharge pattern, is tolerated by most drivers.

Lesser capacities and more variable distribution of delay occurs where demand is not as evenly split among the approaches.

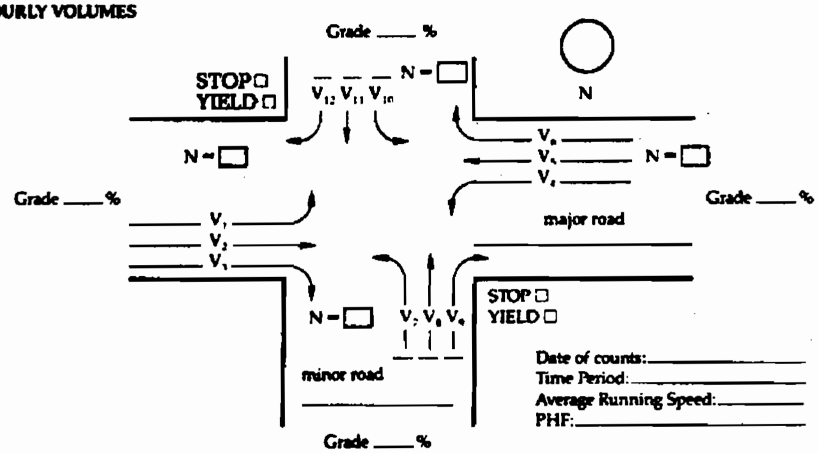
The number of approach lanes also affects the capacity of multiway STOP-controlled intersections. Simultaneous movements from a two-lane approach can occur, increasing the overall capacity. Table B6-5 shows the capacity of four-way STOP-controlled intersections with a 50/50 demand split for a range of approach lane configurations.

Table B6-6 gives volume levels which can be accommodated at four-way STOP-controlled intersections under reasonable operating conditions. Although levels of service for such intersections are not specifically defined, Table B6-6 volumes are approximately indicative of LOS C.

WORKSHEET FOR FOUR-LEG INTERSECTIONS Page 1

Location: ______________________ Name: ______________________

HOURLY VOLUMES



Date of counts: ______________
Time Period: ______________
Average Running Speed: ______________
PHF: ______________

VOLUME ADJUSTMENTS

Movement No.	1	2	3	4	5	6	7	8	9	10	11	12
Volume (v/h)												
Vol. (pc/h), see Table B6-1												

VOLUMES IN PC/H

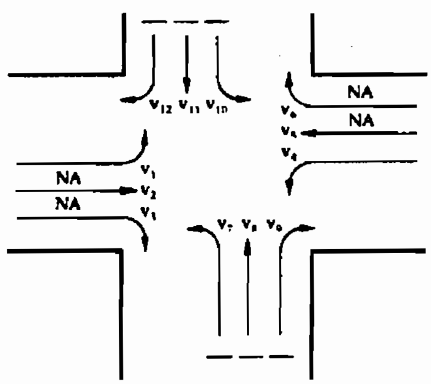


Figure B6-5

Worksheet for Four-Leg Intersections, Page 1

WORKSHEET FOR FOUR-LEG INTERSECTIONS		Page 2
STEP 1: RT From Minor Street	V_9	V_{12}
Conflicting Flows, V_c Critical Gap, T_c (Tab. B6-2) Potential Capacity, c_p (Fig. B6-4) Percent of c_p Utilized Impedance Factor, P (Fig. B6-3) Actual Capacity, c_m	$1/2\ V_3 + V_2 = V_{c9}$ ____ + ____ = ______ v/h ______ (s) c_{p9} = ______ (pc/h) $(v_9/c_{p9}) \times 100$ = ______ % P_9 = ______ $c_{m9} = c_{p9}$ = ______ (pc/h)	$1/2\ V_6 + V_5 = V_{c12}$ ____ + ____ = ______ v/h ______ (s) c_{p12} = ______ (pc/h) $(v_{12}/c_{p12}) \times 100$ = ______ % P_{12} = ______ $c_{m12} = c_{p12}$ = ______ (pc/h)
STEP 2: LT From Major Street	V_4	V_1
Conflicting Flows, V_c Critical Gap, T_c (Tab. B6-2) Potential Capacity, c_p (Fig. B6-4) Percent of c_p Utilized Impedance Factor, P (Fig. B6-3) Actual Capacity, c_m	$V_3 + V_2 = V_{c4}$ ____ + ____ = ______ v/h ______ (s) c_{p4} = ______ (pc/h) $(v_4/c_{p4}) \times 100$ = ______ % P_4 = ______ $c_{m4} = c_{p4}$ = ______ (pc/h)	$V_6 + V_5 = V_{c1}$ ____ + ____ = ______ v/h ______ (s) c_{p1} = ______ (pc/h) $(v_1/c_{p1}) \times 100$ = ______ % P_1 = ______ $c_{m1} = c_{p1}$ = ______ (pc/h)
STEP 3: TH From Minor Street	V_8	V_{11}
Conflicting Flows, V_c Critical Gap, T_c (Tab. B6-2) Potential Capacity, c_p (Fig. B6-4) Percent of c_p Utilized Impedance Factor, P (Fig. B6-3) Actual Capacity, c_m	$1/2V_3+V_2+V_1+V_6+V_5+V_4=V_{c8}$ ____ + ____ + ____ + ____ + ____ + ____ = ____ v/h ______ (s) c_{p8} = ______ (pc/h) $(v_8/c_{p8}) \times 100$ = ______ % P_8 = ______ $c_{m8} = c_{p8} \times P_1 \times P_4$ ______ = ______ × v/h ____ × ______ (pc/h)	$1/2V_6+V_5+V_4+V_3+V_2+V_1=V_{c11}$ ____ + ____ + ____ + ____ + ____ + ____ = ____ v/h ______ (s) c_{p11} = ______ (pc/h) $(v_{11}/c_{p11}) \times 100$ = ______ % P_{11} = ______ $c_{m11} = c_{p11} \times P_1 \times P_4$ ______ = ______ × ______ × ______ (pc/h)
STEP 4: LT From Minor Street	V_7	V_{10}
Conflicting Flows, V_c Critical Gap, T_c (Tab. B6-2) Potential Capacity, c_p (Fig. B6-4) Actual Capacity, c_m	V_{c8} (step 3) $+ V_{11} + V_{12} = V_{c7}$ ____ + ____ + ____ = ____ v/h ______ (s) c_{p7} = ______ (pc/h) $c_{m7} = c_{p7} \times P_1 \times P_4 \times P_{11} \times P_{12}$ ______ = ____ × ____ × ____ × ____ × ____ (pc/h)	V_{c11} (step 3) $+ V_8 + V_9 = V_{c10}$ ____ + ____ + ____ = ____ v/h ______ (s) c_{p10} = ______ (pc/h) $c_{m10} = c_{p10} \times P_4 \times P_1 \times P_8 \times P_9$ ______ = ____ × ____ × ____ × ____ × ____ (pc/h)

Figure B6-6
Worksheet for Four-Leg Intersections, Page 2

WORKSHEET FOR FOUR-LEG INTERSECTIONS — Page 3

SHARED-LANE CAPACITY

$$c_{SH} = \frac{v_l + v_t}{(v_l/c_{ml}) + (v_t/c_{mt})}$$ where 2 movements share a lane

$$c_{SH} = \frac{v_l + v_t + v_r}{(v_l/c_{ml}) + (v_t/c_{mt}) + (v_r/c_{mr})}$$ where 3 movements share a lane

MINOR STREET APPROACH MOVEMENTS 7, 8, 9

Movement	v (pc/h)	c_m (pc/h)	c_{SH} (pc/h)	$c_R = c_{SH} - v$	LOS
7					
8					
9					

MINOR STREET APPROACH MOVEMENTS 10, 11, 12

Movement	v (pc/h)	c_m (pc/h)	c_{SH} (pc/h)	$c_R = c_{SH} - v$	LOS
10					
11					
12					

MAJOR STREET LEFT TURNS 1, 4

Movement	v (pc/h)	c_m (pc/h)	$c_R = c_m - v$	LOS
1				
4				

COMMENTS:

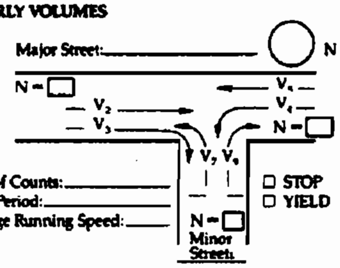
Figure B6-7

Worksheet for Four-Leg Intersections, Page 3

WORKSHEET FOR ANALYSIS OF T-INTERSECTIONS

LOCATION:______________________ NAME:______________________

HOURLY VOLUMES

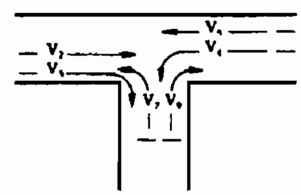


Grade __%

Date of Counts:__________
Time Period:__________
Average Running Speed:__________

PHF:_____ Grade_____%

VOLUMES IN PCPH



VOLUME ADJUSTMENTS

Movement No.	2	3	4	5	7	9
Volume (v/h)						
Vol. (pc/h), see Table B6-1						

STEP 1: RT from Minor Street V_9

Conflicting Flow, V_c	$1/2\ V_3 + V_2 =$ ___ + ___ = ___ v/h (V_{c9})
Critical Gap, T_c, and Potential Capacity, c_p	$T_c =$ ___ s (Table B6-2) $c_{p9} =$ ___(pc/h)(Fig. B6-4)
Actual Capacity, c_m	$c_{m9} = c_{p9} =$ ___

STEP 2: LT From Major Street V_4

Conflicting Flow, V_c	$V_3 + V_2 =$ ___ + ___ = ___ v/h (V_{c4})
Critical Gap, T_c, and Potential Capacity, c_p	$T_c =$ ___ s (Table B6-2) $c_{p4} =$ ___(pc/h)(Fig. B6-4)
Percent of c_p Utilized and Impedance Factor (Fig. B6-3)	$(v_4/c_{p4}) \times 100 =$ ___ $P_4 =$ ___
Actual Capacity, c_m	$c_{m4} = c_{p4} =$ ___ (pc/h)

STEP 3: LT From Minor Street V_7

Conflicting Flow, V_c	$1/2\ V_3+V_2+V_5+V_4 =$ ___ + ___ + ___ + ___ = ___ v/h (V_{c7})
Critical Gap, T_c, and Potential Capacity, c_p	$T_c =$ ___ s (Table B6-2) $c_{p7} =$ ___(pc/h)(Fig. B6-4)
Actual Capacity, c_m	$c_{m7} = c_{p7} \times P_4 =$ ___ $\times$ ___ = ___ (pc/h)

SHARED-LANE CAPACITY

$$SH = \frac{v_7 + v_9}{(v_7/c_{m7}) + (v_9/c_{m9})} \text{ if lane is shared}$$

Movement No.	v (pc/h)	c_m (pc/h)	c_{SH} (pc/h)	c_R	LOS
7					
9					
4					

Figure B6-8

Worksheet for "T" Intersections

B.7 SIGNALIZED INTERSECTIONS AND SIGNAL TIMING

B.7.1 Introduction

This section is a summary of the Ministry manual for Traffic Control Signal Timing and Capacity Analysis at Signalized Intersections (December 1989). A complete description of the MTO methodology for calculating traffic control signal timing and capacity analysis at signalized intersections together with examples is contained in the manual. The manual can be obtained from the Traffic Management and Engineering Office at the MTO. Historically, the MTO has provided guidelines on calculating traffic control signal timing and capacity analysis at signalized intersections through manuals issued from its Head (Downsview) Office. The manual is a complete update of the manual issued in 1976 and was last revised in 1982.

In the manual many significant changes, reflective of current traffic signal timing and capacity analysis methods and traffic characteristics, have been made. The changes include significant changes to the following:

i) Consideration of the Intersection Operating Characteristics for Determining the "Time to Enter Intersection";

ii) Amber Interval Timing;

iii) Pedestrian Interval Timing;

iv) Left Turn Equivalent Factors;

v) Equivalent Vehicles for Trucks and Buses;

vi) Consideration of Peak Hour Factor (for Signal Timing and Capacity Analysis);

vii) Left Turn Capacity Analysis.

Reference is made in the manual to other capacity methods and manuals. After being acknowledged they are then referred to by an abbreviation. These abbreviations and others commonly used throughout this section are listed below:

CCG	-	Canadian Capacity Guide for Signalized Intersections (Urban) (1984)
FHWA	-	Federal Highway Administration
MUTCD	-	Ontario Manual of Uniform Traffic Control Devices
HCM	-	Highway Capacity Manual (1985)
PHF	-	Peak Hour Factor
LOS	-	Level of Service
MTO	-	Ministry of Transportation of Ontario

B.7.2 Considerations for Capacity Analysis

B.7.2.1 Time to Enter Intersection

Field observations have indicated that, if a queue of passenger vehicles is waiting at a red signal, the time required for successive vehicles to enter the intersection when the signal turns green is a function of two time components. Firstly, an initial reaction time or starting delay which results in longer headways between the first vehicles entering the intersection. This produces a low discharge rate at the start of every phase. Secondly, a minimum headway (maximum discharge rate) ranging from approximately 2.0 s (0.5 s discharge rate) for a rural intersection to 1.8 s (0.56 s discharge rate) for an urban/commuter intersection.

The time required for successive vehicles to enter the intersection is shown in Table B7-1.

The data for Rural Conditions are based on "Greenshields's Chart of Headways for Passenger Vehicles"* and are shown for the timing for `X' vehicles in Table B7-4. The data for Urban/Commuter Conditions were obtained from observations (carried out by the MTO in 1986, 1987 and 1988) at intersections operating at capacity in the Metro Toronto area. These data are shown as the timing for `X' vehicles in Table B7-2.

While Tables B7-1 and B7-2 are labelled Rural and Urban/Commuter respectively, these descriptions should not be strictly adhered to. The Urban/Commuter table should be used for any intersection operating daily with aggressive driving characteristics normally associated with large urban areas or commuter traffic. The rural table should be used for any other intersections that are not influenced by commuter traffic.

* Greenshields B.D., Shapiro D., Erickson E.L., Traffic Performance at Urban Street Intersections (Technical Report, Yale University), Bureau of Highway Traffic, New Haven, Connecticut, 1947.

Table B7-1

TIME TO ENTER INTERSECTION

RURAL CONDITIONS			URBAN/COMMUTER CONDITIONS		
Vehicle Position in Queue	**Average Headway (s)**	**Time to Enter Intersection (s)**	**Vehicle Position in Queue**	**Average Headway (s)**	**Time to Enter Intersection (s)**
1	3.8	3.8	1	2.6	2.6
2	3.2	7.0	2	2.3	4.9
3	2.7	9.7	3	2.1	7.0
4	2.3	12.0	4	2.0	8.9
5	2.2	14.2	5	1.9	10.8

Tables B7-1 and B7-2 provide tabulated values for the relationship between the Level of Service (LOS), the average number of vehicles `m' arriving during a cycle and the probability of `X' or fewer vehicles arriving during the cycle. From these tables, phase green plus amber times for various probabilities of vehicle arrivals can be obtained (refer to Section B.7.5).

This method of capacity and signal timing was originally derived for isolated rural intersections; however, experience has proven that this method is also useful for analyzing urban or semi-urban intersections. With the addition of Table B7-1 accurate timing of urban/commuter intersections is possible. This method is not used where left turning vehicles face an advanced green indication, since it is recognized that 2.0 s/v is generally sufficient to complete the turn during the advanced green phase for any number of left turning vehicles.

It is important to note that the maximum discharge rate (i.e., the minimum headways through an intersection) can vary from one geographic area to another. Traffic flow surveys documented in the CCG indicate that discharge flow rates are not constant and that these rates drop after 40 to 45 s of green.

The CCG shows a series of basic saturation flow values for various geographic locations and conditions. As a point of reference, the 2.0-s minimum headway (0.5-s discharge rate) shown in Table B7-1 would yield a saturation flow of 1800 pc/h of green and the 1.8-s minimum headway (0.56 s discharge rate) shown in Table B7-2 would yield a saturation flow of 2016 pc/h green. When specific site information is not available, the Table B7-1 value of 2.0 s (minimum headway) is considered to be useful average representative.

B.7.2.2 Equivalent Vehicle Volumes

In order to determine signal timing, equivalent vehicle volumes (V_e) in terms of passenger cars rather than the actual traffic volumes should be used. Equivalency factors will compensate for commercial vehicles in the traffic flow and for turning movements. The V_e is used to determine the critical lane volume or the critical movement direction for each phase of a cycle.

i) Effects of Commercial Vehicles

Commercial vehicles require more time to clear the intersection than the average passenger car because of their length and generally lower acceleration ability. There are large varieties in length, type and weight-to-power ratio of trucks. According to the latest FHWA sponsored study, the passenger car equivalency factor ranges from 1.1 to 2.5.* An average representation can be assumed; that is, an average truck, a recreational vehicle or a bus is equivalent to 2.0 pc.

*PRC Voorhees (prepared for FHWA), Passenger Car Equivalencies on Urban Arterial Roads, PRC Voorhees, McLean, Virginia, November 23, 1981, p.1.

ii) Effects of Turning Movements

a) Left Turn Movement

Left turning vehicles will take more time to clear the intersection than the straight through vehicles because of the opposing traffic (except in the case of a fully protected left turn operation). They will also block through vehicles in their lane, unless there is a separate left turn lane providing adequate storage for the left turning vehicles. Different methods and considerations are used in estimating the capacity of a shared left turn lane.

For a single lane approach Greenshield concluded that the time required for a left turning vehicle to complete its turn was, on the average, the amount of time required for 1.6 straight through vehicles.** Recent studies have incorporated the effects of the opposing traffic flow and the gap availability through the opposing lanes in determining the left turn factor. The HCM recommends the left turn equivalency factors (f_L) shown in Table B7-2.

Table B7-2

LEFT TURN EQUIVALENCY FACTORS

Opposing Flow Convert to (v/h)			(Left Turn Equivalency Factor to Convert to Through Vehicles, f_L)
0	-	199	1.1
200	-	599	2.0
600	-	799	3.0
800	-	999	4.0
Greater than		999	5.0

The opposing flow is the total volume of opposing through and right turn lane volumes where right turn channelization does not exist.

For a two lane approach with a shared left and through lane a left turn equivalency factor of 4.5 should be used. This is an artificial factor, which attempts to represent the redistribution of through traffic. The operation of a two-lane approach with no left turn lane, usually results in a shift of through vehicles from the shared lane into the curb lane.

The 4.5 factor is used to represent a shift of 75% of the through traffic into the curb lane. This factor provides accurate results when the left turn volume is less than 20% of the total approach volume.

When the left turn volume is greater than 20% of the total approach volume, then the shared lane essentially acts as an exclusive left turn lane and all through vehicles use the curb lane. Capacity and signal timing must therefore be calculated using this condition.

b) Right Turn Movement

Right turn vehicles in a shared lane can also impede the through traffic since they require extra green time to complete their turn. If there is a free flowing right turn channelization or a sufficient length of right turn lane where the turning vehicles do not interfere with the through traffic, no equivalency factor is used, and the right turn volume is not usually considered as part of the critical lane(s) volume. However, at certain locations with an exclusive right turn lane, the right turn volume may be the critical movement, and in that case it would govern the capacity and signal timing calculations.

When there is a through lane without an adequate auxiliary lane for right turns, the applicable equivalency factor is 1.2 through passenger cars for each right turning vehicle.

**Greenshields B.D., Shapiro D., Erickson E.L., Traffic Performance at Urban Street Intersections (Technical Report, Yale University), Bureau of Highway Traffic, New Haven, Connecticut, 1947.

iii) Critical Lane Volume

For capacity and signal timing calculations it is important to identify the critical lane volume for each phase.

For a phase without separate turning lanes on the approaches it is obvious that the direction with the higher equivalent volume per lane is critical. The movement from the other direction is not considered since it will use the same phase time.

When separate turning lanes are used, the equivalent vehicle volume of the turning lane should be compared to the equivalent vehicle volume of the other lanes to determine the critical volume. If the left turns are dominant, a separate left-turn phase operation may be required (refer to Section B.8). It should be noted that when a through volume moves with a concurrent phase (e.g., advanced green), critical movements must be considered (refer to Section B.5) when the green time is calculated for the through phase.

For approaches with two or more through lanes, the equivalent volume of the through lanes will be averaged over the number of through lanes. For intersections without left turn phasing, the critical lane volume for a phase will, therefore, be the highest average lane volume.

B.7.2.3 Random Flow and Poisson Probability

At a signalized intersection, the optimum operation would have all vehicles which arrive during a cycle clearing on the same green interval or on the next green interval. As volumes increase, this operation is not possible since more vehicles arrive in some cycles than can clear on the following green interval. The number of cycles when this occurs determines the operational quality of the intersection. The quality of the traffic flow is measured by the Level of Service.

Since a traffic stream usually follows a random distribution pattern, this random flow can be expressed by the Poisson probability distribution to determine the probability of any number of vehicles arriving during a cycle, based on the average arrivals.

B.7.2.4 Average Arrival Rate

If `m' represents the average number of equivalent vehicles per lane arriving in one cycle for a particular phase, and V_e is the critical equivalent volume for the phase, then:

$$m = \frac{V_e}{n} \div \text{number of cycles per hour}$$

$$m = \frac{V_e}{n} \div \frac{3600}{C}$$

$$\text{Therefore } m = \frac{V_e C}{n \times 3600}$$

Where, n = number of lanes used by the critical volume

C = cycle length in seconds

Average arrivals per cycle m indicate that 50% of the time, more than the average number of vehicles will arrive and if the intersection was only timed to handle average arrivals, a low level of Service would result.

Levels of Service, therefore, are assigned to different probabilities of arrivals as determined by the Poisson distribution. For example:

> If the average arrivals, m, at an approach to a signalized intersection are 10 pc/cycle, the probability (from Poisson) of 15 or less v/cycle arriving is 95% (using Table B7-1 or B7-2). Therefore, if the intersection is timed to handle 15 v/cycle, 95% of the time all arrivals will be able to clear on the same or on the next green interval. Conversely, 5% of the time more than 15 v/cycle will arrive,a backup. This quality of operation at 95% probability is Level of Service `A'.

B.7.2.5 Peak Hour Factor

In signal capacity calculations and in most signal timing calculations the peak hours are used. If the volume over the peak hour is not uniform there may be a period during the hour when the calculated signal timing is not sufficient. The PHF is a measure of short term volume variations over the peak hour. The PHF (for an approach) is defined as the ratio of the total peak hour (approach) volume (V_A peak hour) to the peak flow that occurs during 15 minutes of the peak hour expressed as a hourly flow:

$$\text{PHF} = \frac{V_A\ (\text{peak hour})}{4 \times \text{highest 15-min volume}}$$

The PHF can range from 1.0 (uniform flow) to 0.25 (all volume occurring in one 15-min period). In general, the PHF for an approach of a signalized intersection should average 0.85 and no correction to signal timing is recommended for a PHF at or above 0.80. For a PHF below 0.80 the peak hour equivalent volume (V_e) should be divided by the PHF for capacity and timing calculations.

B.7.3 Level of Service

The MTO methodology for calculating green timing employs the Poisson random probability function. This is based on the concept that vehicles arriving at a signalized intersection will, to a certain degree of probability, be able to clear the intersection during the first green interval encountered upon their arrival. The degree of probability determines the Level of Service.

Figure B7-1 graphically depicts the relationship between the average number of vehicles arriving per cycle m and the green plus amber time required (for a phase) at different LOS. Tables B7-4 and B7-5 tabulate this relationship. The values of m form the main part of Tables B7-4 and B7-5, and are identical for both tables. Table B7-4 shows the green plus amber time requirements for rural conditions, and Table B7-4 shows the green plus amber time requirements for urban/commuter conditions.

Tables B7-4 and B7-5 give the values of green plus amber time for various probabilities of vehicle arrivals (i.e., 95% - LOS `A'; 90% - LOS `B'; 75% -LOS `C'; 60% - LOS `D' and 50% - LOS `E').

B.7.4 Considerations for Signal Timing

B.7.4.1 Discharge Rate and Theoretical Capacity

A traffic stream approaching a signalized intersection during the red interval has to stop, and when the green interval commences, a certain time will elapse before the accelerating vehicles can attain the normal operating speed. The queue discharges rapidly and a few seconds after the start of the green interval, a maximum discharge rate is attained. However, this maximum rate of discharge is not constant for long green times.

B.7.4.2 Lost Capacity Due to Lost Time

Lost capacity occurs at a signalized intersection due to lost time during each phase. Lost time is the difference between the actual phase time and the effectively used portion of the phase time.

Due to a driver's perception and reaction time and a vehicle's acceleration capability, there is a starting delay in each phase at the beginning of the green interval. Also, at the end of the phase there will be an end loss time which consists of the unused portion of amber interval (assumed as 0.5 s) plus the all-red clearance interval. Figures B7-2 and B7-3 are the graphical representation of Tables B7-4 and B7-5 and illustrate the rate of discharge and the lost capacity during the start of a phase. Using Figure B7-2 for rural intersections and Figure B7-3 for urban/commuter intersections, the lost capacity (l_s) for startup can be read from the graphs for various phase green times. For example, for rural intersections (Figure B7-2), the startup lost capacity (l_s) for phase green times of 44 s or greater is 3 v/phase.

By dividing lost capacity by the maximum discharge rate, the lost time can be obtained. The total lost time per hour for a 2-phase signal with equal phase times is shown for various cycle lengths in Figure B7-4.

After determining the startup lost capacity (l_s) per phase from Figures B7-2 or B7-3, cycle lengths (C) can be related to entering volumes using the following formulas:

Rural Intersections

$$C = \frac{2.0\, l_s N + 0.5N + R}{1 - (V_T / 1800)}$$

Urban/Commuter Intersections

$$C = \frac{2.8\, l_s N + 0.5N + R}{1 - (V_T / 2016)}$$

Where:

C = cycle length, s
l_s = lost capacity per phase due to startup, v/phase
N = number of phases
R = total all-red time for all phases per cycle, s
V_T = total entering volume (sum of the vehicles per lane from the critical movements), v/h.

V_T is the capacity for various cycle lengths and this relationship is plotted in Figure B7-5 for rural and urban/commuter intersections. From Figure B7-5 it can be seen that intersection capacity drops substantially when cycle lengths fall below 60 s. In addition, there are only minor increases in capacity between 60-s and 100-s cycles, and there are only minimal increases in capacity when cycle lengths rise above 100 s.

B.7.4.3 Guidelines for Cycle Length Selection

The useful range for cycle lengths is between 50 and 120 s. For a simple four-leg intersection at a low speed urban location where intersecting roads are of 10 to 12 m in width and traffic volume is not heavy, a total cycle length in the range of 50 to 70 s is preferable. Where intersecting roads are wider, necessitating longer pedestrian crossing time, or volumes are heavy, or turning interference is substantial, the cycle time may vary between 60 to 90 s. In the case of a 3 or 4 phase operation, the cycle time may range from 90 to 120 s. These conditions for cycle lengths generally apply only to fixed time signals. Since traffic actuated signals respond to the actual traffic demand, the cycle length can vary continuously between the set minimum and maximum times.

For capacity calculations, a cycle length of 90 s is usually considered optimum, since lost time is approaching a minimum and capacity is approaching a maximum.

B.7.5 Green Interval

To determine the length of a green interval for a through phase it is necessary to identify the critical movement for the phase. The critical movement is the phase movement with the highest average equivalent volume (V_e) per lane or highest average arrival rate per lane m. Using the average arrival rate m for the critical movement (at a specified LOS), the green plus amber time for the phase is obtained from Tables B7-4 or B7-5 (for rural or urban/commuter conditions respectively). To obtain the length of the green interval, the time required for the amber interval (refer to Section B.7.6) must be subtracted from the green plus amber time obtained (from either Table B7-4 or Table B7-5 or Figure B7-1).

It should be noted that even though a through volume may be critical, if it moves with another concurrent phase (e.g., advanced green) the advanced green phase time should be subtracted from the required through phase time. This may make the time required for the opposing movement critical.

Signal timing must satisfy driver expectation. Although, a signal timing calculation may give short green times for certain phases, motorists, however, expect a reasonable length of green indication and the following minimum green times for through phases are recommended:

a) Main Road: 15 to 20 s green (20 s preferred minimum);

b) Cross Road: 7 to 10 s green (10 s preferred minimum).

If pedestrians are present at a signalized intersection, even in small numbers, their minimum safe crossing needs must be accommodated. Therefore, at fixed time intersections the minimum green time for a phase (with permissive pedestrian movements) must accommodate the pedestrian timing. The timing of pedestrian intervals is described in Section B.7.7.

B.7.6 Clearance Period

When a vehicle is approaching a signalized intersection, the driver can be faced with a dilemma at the onset of the amber indication. A decision must be made whether to stop or go through the intersection. If the decision is to stop and provided that the driver has allowed for a sufficient stopping distance before the intersection, then the length of clearance period will have no effect on the stopping vehicle. However, if the motorist decides to proceed, then the clearance period must be long enough to take the vehicle through the intersection before the conflicting traffic receives a green indication.

The clearance period is characterized by an amber warning indication followed by an all-red clearance indication. The amber warning indication tells an approaching driver that the right-of-way is about to be assigned to conflicting traffic, and the all-red clearance indication provides time for vehicles in the intersection to clear before the green indication is displayed to conflicting traffic. When the amber and all-red intervals are not properly timed, some drivers may be forced to panic-stop or go through the intersection as the conflicting traffic is released. Thus, inadequate timing of the clearance period could cause rear end and angle collisions.

The required clearance period for any through movement phase is related to the approach operating speed, the driver perception and reaction time, the crossing width of the intersection and the average deceleration rate of the motor vehicle.

The total clearance period is separated into two distinctive parts (intervals) and expressed by the following formula for level road conditions:

$$y + r = t + \frac{V}{2a} + 3.6\,\frac{W+1}{V}$$

(refer to Table B7-6 for definition of variables).

The first part of the equation (t + V/2a) represents the amber interval indicating that the right-of-way is about to be changed. This interval is obtained by dividing the Stopping Sight Distance (SSD = Vt + V^2/2a) by the approach speed (V). The amber interval (y) provides sufficient time for the approaching driver travelling at the approach speed to travel the stopping distance.

The second part of the equation $3.6(W+1)/V$ represents the all-red interval (r) required to provide a safe passage across the intersection for vehicles entering the intersection at or near the end of the amber interval. It is recommended that all-red intervals be used at all signalized intersections.

The proper timing of the clearance period is crucial for the safe and efficient operation of a signalized intersection. If the clearance period is set too short, the driver may not be able to proceed through the intersection before the conflicting traffic is released. If the amber interval is too long, drivers tend to treat this interval as an extension of green time. This can create driver disrespect for the amber indication resulting in a higher incidence of accidents.

In the equation for the amber interval the perception-reaction time has been increased to 1.8 s from the 1.0 s currently in use in the Signal Chapter of the Ontario Manual of Uniform Traffic Control Devices (MUTCD).

In the calculation of minimum stopping sight distances on a free flow highway a perception- reaction time of 2.5 s is used. This is based on an unexpected roadway condition that requires the vehicle to stop. A long perception time is needed to determine that a stop condition exists. Once the stop condition is perceived the decision to stop has been taken and a normal brake reaction time would follow the decision.

At a signalized intersection, when an amber is displayed, the perception time is shorter since the driver is aware that there is a signal ahead. However, the driver must then decide whether to stop or go based on the vehicle speed and the distance to the intersection. This decision may not be instantaneous and brake reaction time cannot start until the decision to stop is taken.

Studies have shown that the 85th percentile perception-reaction time for a driver confronted by an amber indication is 1.8 s.* This indicates that there is more hesitation than had been assumed in the previously accepted 1.0 s perception-reaction time.

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*Taoka G.T., Brake Reaction Times of Unalerted Drivers, ITE Journal, Washington, D.C., March 1989. p.21.

Sabra Z.A., Driver Response to Active Advance Warning Signs at High Speed Signalized Intersections, Department of Civil Engineering West Virginia University, Morgantown, W.V., October 1985, Report No. FHWA/RD - 86/130, p.38.

Gordon D.A., McGee H.W., Hooper K.G., Driver Characteristics Impacting Highway Design and Operations, Public Roads, June 1984, p. 15.

Chong M.S., Messer C.J., Santiago A., Evaluation of Engineering Factors Affecting Traffic Signal Change Interval, Transportation Research Record No. 956, 1984, p. 18.

Hooper K.G., McGee H.W., Driver Perception-Reaction Time: Are Revisions to Current Specification Values in Order?, Transportation Research Record No. 904, 1983, p. 28.

Table B7-6 shows the required amber and all-red intervals for different intersection widths and approach speeds for zero percent grade based on the total clearance period formula. It should be noted that when the intersection approach is on a grade, the total clearance period formula should incorporate the uphill (+) or downhill (-) grade.

B.7.7 Pedestrian Interval

If pedestrians are present at a signalized intersection, even in small numbers, their minimum safe crossing needs must be accommodated. The duration of a phase that will be utilized by pedestrians should provide a suitable starting time plus sufficient time for pedestrians to clear the conflict zone prior to the release of a conflicting traffic movement.

The walking speed of pedestrians usually varies between 1.0 and 1.25 m/s. A normal walking speed of 1.25 m/s is generally used for pedestrian timing together with a minimum starting time of 5 s. This provides a total required crossing time of $5 + 0.8Wc$ s, where Wc is the pedestrian crossing distance in metres.

The total crossing time is split up into a 'walk' time shown by a 'walking man' symbol and a pedestrian clearance time which can consist of a flashing and solid 'don't walk' symbol ('Hand Outline').

To provide a suitable pedestrian clearance time, 50% of the crossing distance is used as a minimum time.

It has been observed that longer pedestrian clearance times tend to increase pedestrian disobedience of the pedestrian indications. Many pedestrians walk at a speed greater than 1.25 m/s, and when the flashing 'don't walk' indication is displayed for a long period of time while the vehicle indication is still green, these pedestrians know that they can still cross before the vehicle amber indication comes on. Once pedestrians start to rely on the vehicle heads instead of the pedestrian heads, the purpose of the pedestrian clearance is lost.

If compliance with the pedestrian signal heads is expected, pedestrians must be provided with a meaningful indication. This can be achieved by keeping the pedestrian clearance interval as short as possible, while still providing a proper warning and/or crossing time for pedestrians.

For pedestrians who normally walk at less than 1.25 m/s the use of a pedestrian clearance time based on 50% of the crossing distance will encourage these pedestrians to return to the curb if they have just started to cross the intersection or to increase their walking speed if they are in the crosswalk.

Therefore, the pedestrian clearance time (P_c) is:

$$P_c = \frac{0.50\ W_c}{1.25} = 0.4\ W_c$$

Where:

P_c = pedestrian clearance time, s. (can include vehicle amber time)

W_c = longest length of crossing distance, from curb to far side edge of conflict area, m.

The start of the pedestrian clearance time is normally displayed by a flashing `Don't Walk' indication which changes to a solid `Don't Walk' indication at the start of vehicle amber. A minimum of 5 s flashing `Don't Walk' indication is recommended whenever pedestrian heads are used. Therefore, pedestrian clearances which include the vehicle amber time should never be less than 5 s plus the vehicle amber time.

The pedestrian walk time (Walk) is:

$$Walk = 5 + \frac{0.5\ W_c}{1.25} = 5 + 0.4\ W_c$$

Unless there are unusual conditions at the intersection, the minimum pedestrian clearance time of 0.4 W_c should provide an acceptable operation. When the vehicle green plus amber time is greater than the minimum pedestrian phase time, any additional time should be added to the pedestrian walk time.

Pedestrian clearance times greater than 0.4 W_c may be required at intersections where there are school crossings, a predominance of elderly pedestrians or disabled people. These situations should be treated on an individual basis.

B.7.8 Left Turn Manoeuvres

B.7.8.1 Left Turn Lanes

The left turn manoeuvre is one of the main factors that affects the capacity of a signalized intersection and the safety of the traffic operation. When the through and the left turning traffic volume is light, the left turning vehicles can pick their way through available gaps in the opposing traffic flow. However, when the traffic volume increases in both directions, left turning drivers not only experience difficulty in finding suitable gaps to complete their turn, but they can also impede the through traffic behind them if left turn lanes do not exist at the intersection. Drivers intending to go through the intersection and trapped behind a left turning queue, may commit hazardous manoeuvres such as abrupt stops or sudden lane changes. The left turning drivers could also become impatient after enduring long delays and may try to complete their turn through a gap shorter than a safe gap. Both intersection capacity and operational safety suffer as a result.

Signalized intersections do not operate effectively without left turn lanes. A signal is a restrictive traffic control device that only permits traffic movement during a specific green period. If through vehicles are delayed or stopped by a left turning vehicle, the capacity of the intersection to handle the through volume is severely hampered. For main street volumes, left turn lanes are a necessity.

B.7.8.2 Warrants for Left Turn Phasing

The capacity of a left turn lane can be calculated as (1400 G/C - V_o) (see below for definition of variables). However, this formula fails to recognize the impact of the opposing traffic in more than one lane. Therefore, adjustments are needed if there is more than one lane of opposing traffic. Research conducted in major Canadian cities (Toronto, Calgary, Edmonton) identified different gap sizes and acceptance probabilities as a function of the number of opposing lanes.* It was established that longer gaps are needed to cross more lanes, but the probability of accepting the gaps increased with the number of the opposing lanes. The required gap sizes and the probability of accepting gaps as a function of the number of opposing lanes were converted into modifying factors (f) as indicated in Table B7-3:

Table B7-3

MODIFYING FACTORS

Number of Opposing Flow Lanes	1	2	3	4
Modifying Factor, f	1.0	0.625	0.5	0.44

The opposing volume (V_o) should be adjusted by the modifying factor according to the number of opposing lanes. Substituting the modifying factor into the equation (1400 G/C - V_o) and adding two vehicles per cycle (Lt_a) turning left on amber, the equation for the capacity (c_{Lt}) of a separate left turn lane is as follows:

$$c_{Lt} = 1400\ G/C - (f)\ V_o + Lt_a$$

Where:

- c_{Lt} = the capacity of the separate left turn lane during the "permissive" stage of the phase, v/h.
- f = modifying factor for the number of opposing flow lanes.
- V_o = total opposing flow, v/h (including through lanes, shared lanes and right turn lanes where right turn channelization does not exist).
- C = cycle length, s.
- G = green time (interval) for the opposing flow, s.
- Lt_a = 7200/C, v/h; number of vehicles turning left on amber per hour, (assuming 2 vehicles per cycle turn left on amber from the separate left turn lane).

*Teply S., Richardson D., Schnablegger J. Stephenson B., Canadian Capacity Guide for Signalized Intersections (First Edition), ITE District 7 - Canada, Toronto, Ontario, February 1984, p. 17-18.

Transportation Research Board (TRB), Special Report No. 209 Highway Capacity Manual, TRB, National Research Council, Washington, D.C., 1985, pp. 9-22.

The above formula should be used, to determine if and when a separate left turn phase is warranted. If the calculated value of c_{Lt} is less than the actual number of left turning vehicles, then a separate left turn phase is warranted. If the opposing and the left turning traffic is mixed with trucks, the volumes in the formula should be adjusted by the truck percentages.

Figure B7-6 are nomographs of the formula. Using Figure B7-6 the left turn lane capacity can be determined when the truck percentages in the opposing and the left turning traffic are known.

B.7.8.3 Left Turn Phasing

Types of Left Turn Movements at Signalized Intersections

Permissive Left Turn:

The left turning driver is permitted to turn during the normal green signal display and can complete the turn if adequate gaps occur in opposing traffic. The driver must yield to opposing traffic and pedestrians legally crossing the roadway. The left turning vehicle can clear the intersection on the normal amber indication after yielding to any opposing through vehicles clearing the intersection.

Protected Left Turn:

No conflicting traffic is permitted to proceed during a protected left turn movement. The left turning driver is given a signal display which provides right-of-way for the left turning vehicles over opposing traffic. Pedestrians are prohibited from crossing the path of the left turning vehicle during the protected left turn movement.

Simultaneous Left Turns:

The term simultaneous left turn is used when left turns from opposing directions are allowed to make their turn at the same time during a protected left turn movement.

Types of Left Turn Phasing

Advanced Green:

This phasing gives a protected-permissive left turn movement. The left turning vehicles in one direction are first given a protected phase. The associated through and right turning vehicles are also allowed to proceed during the protected left turn phase. After the protected phase terminates the opposing traffic is released with a normal green signal display. The left turning vehicles are still permitted to turn, however they must yield to any opposing traffic.

Extended Green:

This phasing gives a permissive-protected left turn movement. Left turning vehicles are first permitted to turn after yielding to opposing vehicles during a normal green signal display. They are then provided with a protected left turn phase in one direction after the opposing approach has been terminated with an amber and red signal display. The associated through and right turn movements are allowed to proceed during the protected left turn phase.

To avoid trapping a left turning vehicle, this type of phasing can only be used at locations where there is no opposing left turn movement; for example, at "T" intersections and at Four-Way intersections where the opposing left turn movement is prohibited.

Protected/Permissive Simultaneous Left Turn:

This phasing gives left turning vehicles from opposing directions a protected left turn phase at the same time. No other conflicting vehicles are allowed to enter the intersection during the simultaneous protected left turn phase. After the simultaneous protected left turn phase has terminated, the left turning vehicles are permitted to turn through opposing traffic if adequate gaps are available.

Fully-Protected Simultaneous Left Turn:

This operation provides left-turning vehicles with their own traffic control signal head. Left turning vehicles from opposing directions are given a left turn indication at the same time. No other conflicting vehicles are allowed to enter the intersection during the fully-protected simultaneous left-turn phase. The left turn movements are terminated with their own clearance displays and are not permitted to proceed when the opposing through traffic movements are given green indications.

Separate Phasing:

The phasing allows one approach to the intersection to proceed while the traffic on all other approaches is stopped. All movements on the separate phase approach including left turns are permitted to proceed through the intersection.

Other Types of Left Turn Phasing

Permissive/Protected Simultaneous:

With this type of phasing the left turn movements are permitted to turn through adequate gaps in opposing traffic during the normal green indication and are then given a protected simultaneous left turn phase after the normal green indication has terminated.

To avoid trapping a left turning vehicle in the intersection the normal greens for both opposing directions always have to terminate together. At an actuated intersection, this restriction severely reduces the phasing flexibility since a protected left turn movement in one direction cannot be displayed early even if there is no opposing through traffic.

Another inefficiency with the operation is the fact that the left turning vehicles must first stop during the green termination and then proceed to make their turn on their protected phase.

Lagging Fully-Protected Simultaneous Left Turn:

This operation is similar to the description for "Fully-Protected Simultaneous Left Turn" except the left turn movements are given their protected phase after the through traffic phase. There are no operational restrictions or inefficiencies with this operation. However, it is seldom used since left turn indications are normally brought on before the through traffic indications.

B.7.8.4 Timing Considerations for Left Turn Phasing

i) Protected/Permissive Left Turn Operation

When an advanced green phase (either flashing green or solid green signal and green arrow) or a protected/permissive simultaneous left turn phase is used, the following procedure should be followed when calculating the left turn phase time:

It is not necessary to provide a high LOS for this type of left turn phasing, since left turning vehicles may have the opportunity to complete their movement during the permissive stage. The phase time for the left turning vehicles from the left turn storage lane should be calculated on the basis of the time required to clear the average arrivals at 2 s per vehicle, regardless of what LOS is being used for the other movements. The signal timing for the left turn green time (g_{Lt}) is calculated as follows:

$g_{Lt} = (m_{Lt} - 2) \times 2$ s, where:

$m_{Lt} = \frac{V_{Le} \times C}{3600}$ = left turn arrival rate, pc/cycle/lane

and V_{Le} = equivalent volume of left turning vehicles, pc/h

C = cycle length, s.

Since, on the average, two left turning vehicles can turn during the amber interval, the left turn arrival rate (m_{Lt}) is reduced by two. The reduced number of arrivals is multiplied by two, 2 s being the average headway for left turning vehicles. A minimum length of green for a left turn phase is 5.0 s.

A green clearance or an amber arrow clearance of 1.5 to 2.5 s must follow the left turn green before the opposing traffic is released. An all-red of 1.0 to 1.5 s may be used after the amber arrow if additional clearance is required.

ii) Fully-Protected Left Turn Operation

Since this is a full-phase operation, the left turn green time should be obtained from Table B7-4 or Table B7-5. The LOS applicable is the same as for the other movements.

A nominal amber of 3 s is used after the left turn green time to warn drivers that they are losing the fully protected left turn movement, followed by a 1.5 to 2.0 s all-red to complete the clearance of any left turning vehicles in the intersection.

B.7.8.5 Length of Left Turn Storage Lane

The left turn storage lane should accommodate all left turn arrivals during a cycle. To reduce the possibility of left turn vehicles filling up the storage lane and spilling out into the through lane, the storage lane length is calculated to store the average arrivals 95% of the time (Level of Service "A"). An average length of 7.5 m per passenger car is used in calculating the storage requirements.

The number of vehicles that will arrive 95% of the time based on the average arrival rate per cycle is obtained from Table B7-4 or Table B7-5. This number is multiplied by 7.5 to obtain the left turn storage lane length in metres.

The storage lane length for various cycle times and equivalent left turn volumes together with the green time required for the protected/permissive left turn phase is plotted in Figure B7-7.

Table B7-4

VEHICLE ARRIVAL RATES
AND GREEN PLUS AMBER TIMES FOR RURAL INTERSECTIONS

LEVEL OF SERVICE					"X" VEHICLES	GREEN* PLUS AMBER (s)
A (95%)	B (90%)	C (75%)	D (60%)	E (50%)		
0.0 - 0.3	0.2 - 0.5	0.3 - 1.3	0.5 - 1.3	0.7 - 1.6	1	3.8
0.4 - 0.8	0.6 - 1.1	1.0 - 1.7	1.4 - 2.2	1.7 - 2.6	2	7.0
0.9 - 1.3	1.2 - 1.7	1.8 - 2.5	2.3 - 3.1	2.7 - 3.6	3	9.7
1.4 - 1.9	1.8 - 2.4	2.6 - 3.3	3.2 - 4.0	3.7 - 4.6	4	12.0
2.0 - 2.8	2.5 - 3.1	3.4 - 4.2	4.1 - 5.0	4.7 - 5.6	5	14.2
2.9 - 3.2	3.2 - 3.8	4.3 - 5.0	5.1 - 6.0	5.7 - 6.6	6	16.4
3.3 - 3.9	3.9 - 4.6	5.1 - 5.9	6.1 - 6.9	6.7 - 7.6	7	18.6
4.0 - 4.6	4.7 - 5.4	6.0 - 6.8	7.0 - 7.9	7.7 - 8.6	8	20.8
4.7 - 5.4	5.5 - 6.2	6.9 - 7.7	8.0 - 8.8	8.7 - 9.6	9	23.0
5.5 - 6.1	6.3 - 7.0	7.8 - 8.6	8.9 - 9.8	9.7 - 10.6	10	25.1
6.2 - 6.9	7.1 - 7.8	8.7 - 9.5	9.9 - 10.8	10.7 - 11.6	11	27.2
7.0 - 7.7	7.9 - 8.6	9.6 - 10.4	10.9 - 11.7	11.7 - 12.6	12	29.3
7.8 - 8.4	8.7 - 9.4	10.5 - 11.3	11.8 - 12.7	12.7 - 13.6	13	31.4
8.5 - 9.2	9.5 - 10.3	11.4 - 12.2	12.8 - 13.7	13.7 - 14.6	14	33.5
9.3 - 10.0	10.4 - 11.1	12.3 - 13.1	13.8 - 14.6	14.7 - 15.6	15	35.6
10.1 - 10.8	11.2 - 11.9	13.2 - 14.0	14.7 - 15.6	15.7 - 16.6	16	37.7
10.9 - 11.6	12.0 - 12.8	14.1 - 14.9	15.7 - 16.6	16.7 - 17.6	17	39.8
11.7 - 12.4	12.9 - 13.6	15.0 - 15.9	16.7 - 17.6	17.7 - 18.6	18	41.9
12.5 - 13.2	13.7 - 14.5	16.0 - 16.9	17.7 - 18.5	18.7 - 19.6	19	44.0
13.3 - 14.0	14.6 - 15.3	17.0 - 17.8	18.6 - 19.5	19.7 - 20.6	20	46.0
14.1 - 14.9	15.4 - 16.2	17.9 - 18.7	19.6 - 20.5	20.7 - 21.6	21	48.0
15.0 - 15.7	16.3 - 17.0	18.8 - 19.6	20.6 - 21.5	21.7 - 22.6	22	50.0
15.8 - 16.5	17.1 - 17.9	19.7 - 20.5	21.6 - 22.4	22.7 - 23.6	23	52.0
16.6 - 17.4	18.0 - 18.8	20.6 - 21.4	22.5 - 23.4	23.7 - 24.6	24	54.0
17.5 - 18.2	18.9 - 19.7	21.5 - 22.4	23.5 - 24.4	24.7 - 25.6	25	56.0
18.3 - 19.0	19.8 - 20.6	22.5 - 23.3	24.5 - 25.4	25.7 - 26.6	26	58.0
19.1 - 19.9	20.7 - 21.5	23.4 - 24.3	25.5 - 26.3	26.7 - 27.6	27	60.0
20.0 - 20.7	21.6 - 22.3	24.4 - 25.2	26.4 - 27.3	27.7 - 28.6	28	62.0
20.8 - 21.6	22.4 - 23.2	25.3 - 26.2	27.4 - 28.3	28.7 - 29.6	29	64.0
21.7 - 22.4	23.3 - 24.1	26.3 - 27.1	28.4 - 29.3	29.7 - 30.6	30	66.0
22.5 - 23.3	24.2 - 25.0	27.2 - 28.0	29.4 - 30.3	30.7 - 31.6	31	68.0
23.4 - 24.2	25.1 - 25.9	28.1 - 29.0	30.4 - 31.2	31.7 - 32.6	32	70.0
24.3 - 25.1	26.0 - 26.8	29.1 - 29.9	31.3 - 32.2	32.7 - 33.6	33	72.0
26.2 - 25.9	26.9 - 27.6	30.0 - 30.9	32.3 - 33.2	33.7 - 34.6	34	74.0
26.0 - 26.7	27.7 - 28.5	31.0 - 31.8	33.3 - 34.2	34.7 - 35.6	35	76.0

*Minimum Green: Through Phasing - 7 to 10 s (Minor Rd.)
15 to 20 s (Major Rd.)
Left Turn Phasing - 5 s

Notes:
(1) Relationship between the average and maximum arrival rates is based on the Poisson Distribution.
(2) Relationship between arrival rates and phase times is based on the Greenshield's Chart of Headways for passenger cars.
(3) Each truck or bus is equivalent to 2.0 passenger cars.

Table B7-5

VEHICLE ARRIVAL RATES AND GREEN PLUS AMBER TIMES FOR URBAN/COMMUTER INTERSECTIONS

LEVEL OF SERVICE					"X" VEHICLES	GREEN* PLUS AMBER (s)
A (95%)	B (90%)	C (75%)	D (60%)	E (50%)		
0.0 - 0.3	0.2 - 0.5	0.3 - 1.3	0.5 - 1.3	0.7 - 1.6	1	2.6
0.4 - 0.8	0.6 - 1.1	1.0 - 1.7	1.4 - 2.2	1.7 - 2.6	2	4.9
0.9 - 1.3	1.2 - 1.7	1.8 - 2.5	2.3 - 3.1	2.7 - 3.6	3	7.0
1.4 - 1.9	1.8 - 2.4	2.6 - 3.3	3.2 - 4.0	3.7 - 4.6	4	8.9
2.0 - 2.8	2.5 - 3.1	3.4 - 4.2	4.1 - 5.0	4.7 - 5.6	5	10.8
2.9 - 3.2	3.2 - 3.8	4.3 - 5.0	5.1 - 6.0	5.7 - 6.6	6	12.7
3.3 - 3.9	3.9 - 4.6	5.1 - 5.9	6.1 - 6.9	6.7 - 7.6	7	14.6
4.0 - 4.6	4.7 - 5.4	6.0 - 6.8	7.0 - 7.9	7.7 - 8.6	8	16.5
4.7 - 5.4	5.5 - 6.2	6.9 - 7.7	8.0 - 8.8	8.7 - 9.6	9	18.4
5.5 - 6.1	6.3 - 7.0	7.8 - 8.6	8.9 - 9.8	9.7 - 10.6	10	20.2
6.2 - 6.9	7.1 - 7.8	8.7 - 9.5	9.9 - 10.8	10.7 - 11.6	11	22.0
7.0 - 7.7	7.9 - 8.6	9.6 - 10.4	10.9 - 11.7	11.7 - 12.6	12	23.8
7.8 - 8.4	8.7 - 9.4	10.5 - 11.3	11.8 - 12.7	12.7 - 13.6	13	25.6
8.5 - 9.2	9.5 - 10.3	11.4 - 12.2	12.8 - 13.7	13.7 - 14.6	14	27.4
9.3 - 10.0	10.4 - 11.1	12.3 - 13.1	13.8 - 14.6	14.7 - 15.6	15	29.2
10.1 - 10.8	11.2 - 11.9	13.2 - 14.0	14.7 - 15.6	15.7 - 16.6	16	31.0
10.9 - 11.6	12.0 - 12.8	14.1 - 14.9	15.7 - 16.6	16.7 - 17.6	17	32.8
11.7 - 12.4	12.9 - 13.6	15.0 - 15.9	16.7 - 17.6	17.7 - 18.6	18	34.6
12.5 - 13.2	13.7 - 14.5	16.0 - 16.9	17.7 - 18.5	18.7 - 19.6	19	36.4
13.3 - 14.0	14.6 - 15.3	17.0 - 17.8	18.6 - 19.5	19.7 - 20.6	20	38.2
14.1 - 14.9	15.4 - 16.2	17.9 - 18.7	19.6 - 20.5	20.7 - 21.6	21	40.0
15.0 - 15.7	16.3 - 17.0	18.8 - 19.6	20.6 - 21.5	21.7 - 22.6	22	41.8
15.8 - 16.5	17.1 - 17.9	19.7 - 20.5	21.6 - 22.4	22.7 - 23.6	23	43.7
16.6 - 17.4	18.0 - 18.8	20.6 - 21.4	22.5 - 23.4	23.7 - 24.6	24	45.6
17.5 - 18.2	18.9 - 19.7	21.5 - 22.4	23.5 - 24.4	24.7 - 25.6	25	47.5
18.3 - 19.0	19.8 - 20.6	22.5 - 23.3	24.5 - 25.4	25.7 - 26.6	26	49.4
19.1 - 19.9	20.7 - 21.5	23.4 - 24.3	25.5 - 26.3	26.7 - 27.6	27	51.3
20.0 - 20.7	21.6 - 22.3	24.4 - 25.2	26.4 - 27.3	27.7 - 28.6	28	53.2
20.8 - 21.6	22.4 - 23.2	25.3 - 26.2	27.4 - 28.3	28.7 - 29.6	29	55.1
21.7 - 22.4	23.3 - 24.1	26.3 - 27.1	28.4 - 29.3	29.7 - 30.6	30	57.0
22.5 - 23.3	24.2 - 25.0	27.2 - 28.0	29.4 - 30.3	30.7 - 31.6	31	58.9
23.4 - 24.2	25.1 - 25.9	28.1 - 29.0	30.4 - 31.2	31.7 - 32.6	32	60.8
24.3 - 25.1	26.0 - 26.8	29.1 - 29.9	31.3 - 32.2	32.7 - 33.6	33	62.7
26.2 - 25.9	26.9 - 27.6	30.0 - 30.9	32.3 - 33.2	33.7 - 34.6	34	64.6
26.0 - 26.7	27.7 - 28.5	31.0 - 31.8	33.3 - 34.2	34.7 - 35.6	35	66.5

* Minimum Green: Through Phasing - 7 to 10 s (Minor Rd.)
15 to 20 s (Major Rd.)
Left Turn Phasing - 5 s

Notes:
(1) Relationship between the average and maximum arrival rates is based on the Poisson Distribution.
(2) Relationship between arrival rates and phase times is based on the Greenshield's Chart of Headways for passenger cars.
(3) Each truck or bus is equivalent to 2.0 passenger cars.

Table B7-6

CLEARANCE PERIOD

Operating Speed km/h	25	30	35	40	45	50	60	70	80	90	100	110	120
Amber Warning Interval (s)	3.0	3.2	3.4	3.6	3.8	4.1	4.5	5.0	5.4	5.9	6.3	6.8	7.3
Clearing Distance (m) (W + l)	**ALL RED CLEARANCE INTERVAL (s)**												
12.0	1.7	1.4	1.2	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
13.5	1.9	1.6	1.4	1.2	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
15.0	2.2	1.8	1.5	1.4	1.2	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0
16.5	2.4	2.0	1.7	1.5	1.3	1.2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
18.0	2.6	2.2	1.9	1.6	1.4	1.3	1.1	1.0	1.0	1.0	1.0	1.0	1.0
19.5	2.8	2.3	2.0	1.8	1.6	1.4	1.2	1.0	1.0	1.0	1.0	1.0	1.0
21.0	3.0	2.5	2.2	1.9	1.7	1.5	1.3	1.1	1.0	1.0	1.0	1.0	1.0
22.5	3.2	2.7	2.3	2.0	1.8	1.6	1.4	1.2	1.0	1.0	1.0	1.0	1.0
24.0	3.5	2.9	2.5	2.2	1.9	1.7	1.4	1.2	1.1	1.0	1.0	1.0	1.0
25.5	3.7	3.1	2.6	2.3	2.0	1.8	1.5	1.3	1.1	1.0	1.0	1.0	1.0
27.0	3.9	3.2	2.8	2.4	2.2	1.9	1.6	1.4	1.2	1.1	1.0	1.0	1.0
28.5	4.1	3.4	2.9	2.6	2.3	2.1	1.7	1.5	1.3	1.1	1.0	1.0	1.0
30.0	4.3	3.6	3.1	2.7	2.4	2.2	1.8	1.5	1.4	1.2	1.1	1.0	1.0
31.5	4.5	3.8	3.2	2.8	2.5	2.3	1.9	1.6	1.4	1.3	1.1	1.0	1.0
33.0	4.8	4.0	3.4	3.0	2.6	2.4	2.0	1.7	1.5	1.3	1.2	1.1	1.0
34.5	5.0	4.1	3.5	3.1	2.8	2.5	2.1	1.8	1.6	1.4	1.2	1.1	1.0
36.0	5.2	4.3	3.7	3.2	2.9	2.6	2.2	1.9	1.6	1.4	1.3	1.2	1.1
37.5	5.4	4.5	3.9	3.4	3.0	2.7	2.3	1.9	1.7	1.5	1.4	1.2	1.1
39.0	5.6	4.7	4.0	3.5	3.1	2.8	2.3	2.0	1.8	1.6	1.4	1.3	1.2
40.5	5.8	4.0	4.2	3.6	3.2	2.9	2.4	2.1	1.8	1.6	1.5	1.3	1.2
42.0	6.0	5.0	4.3	3.8	3.4	3.0	2.5	2.2	1.9	1.7	1.5	1.4	1.3
43.5	6.3	5.2	4.5	3.9	3.5	3.1	2.6	2.2	2.0	1.7	1.6	1.4	1.3
45.0	6.5	5.4	4.6	4.1	3.6	3.2	2.7	2.3	2.0	1.8	1.6	1.5	1.3

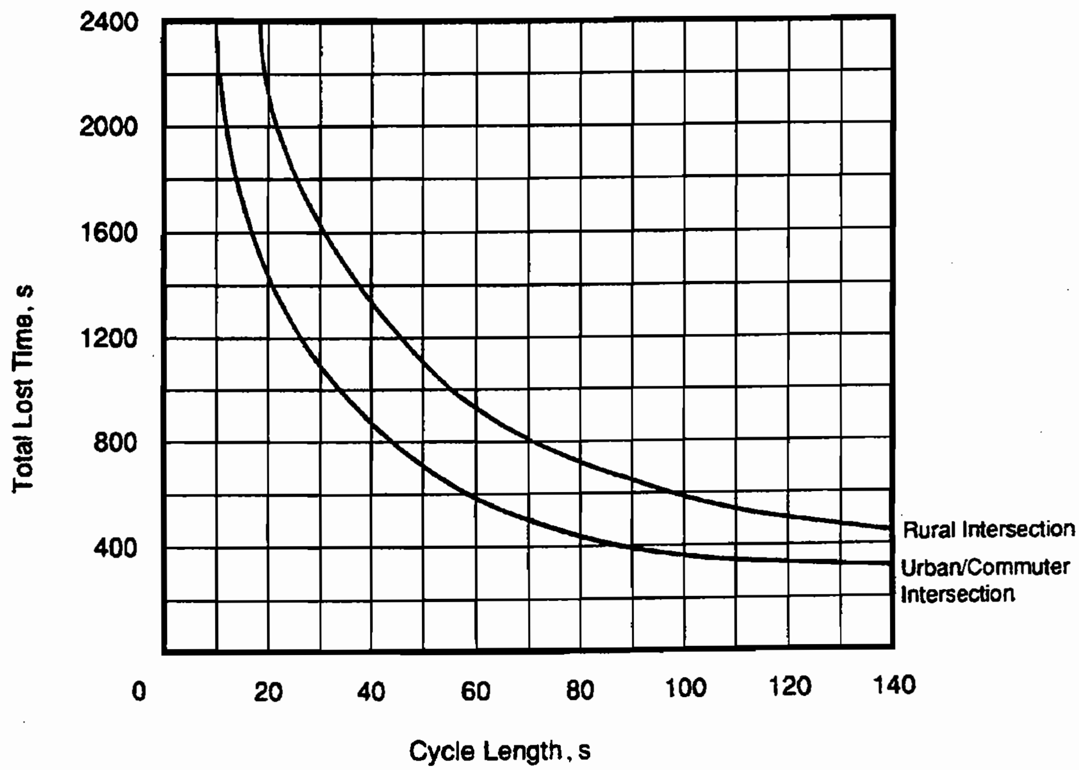
amber interval all red interval

Clearance Period :

$$y + r = t + \frac{V}{2a} + 3.6\,\frac{W + 1}{V}$$

Where:
- y - amber interval, s
- r - all-red interval, s
- t - 1.8 s, perception-reaction time
- V - approach operating speed, km/h
- a - average deceleration rate; 11 km/h/s
- l - 6.0 m, length of the average passenger vehicle
- W - width of intersecting road to be crossed from near side stop bar to far side curb line, m
- 3.6 - conversion factor, to convert km/h into m/s

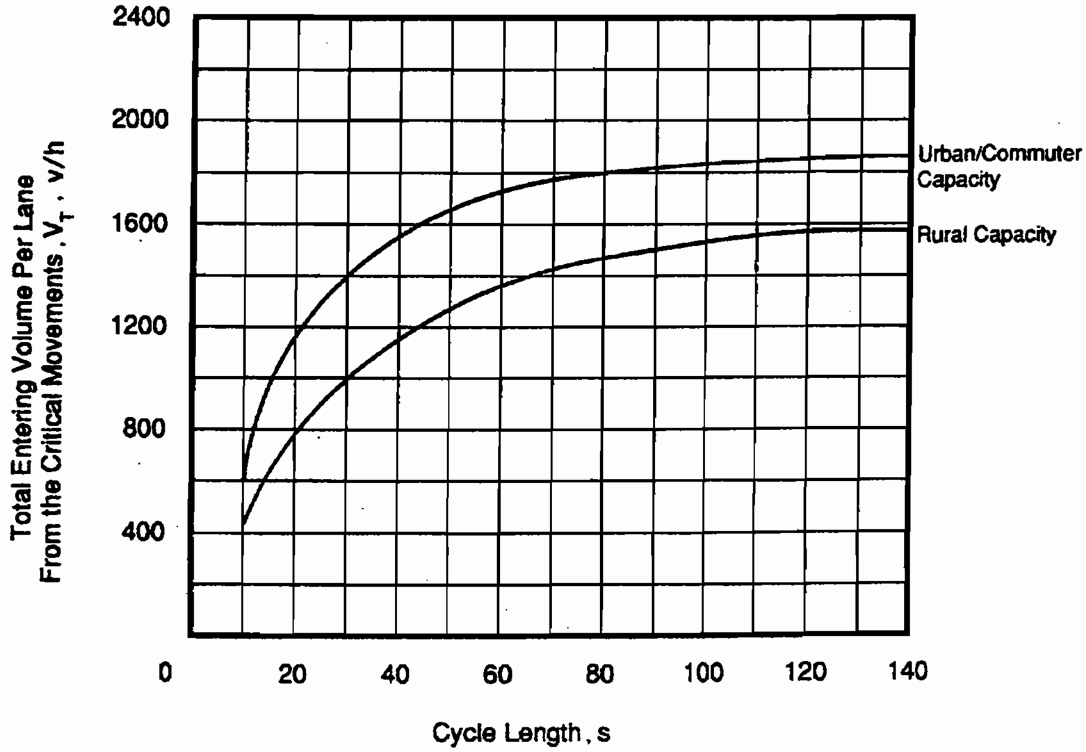
Notes:
(1) Minimum recommended length of amber interval is 3.0 s.
(2) Minimum recommended length of all-red interval is 1.0 s.
(3) This table is not to be used for left turn clearances.
(4) When the intersection approach is on a grade, the total clearance period formula should incorporate the uphill (+) or downhill (-) grade.



Note: Assume 2 Phase Signal Operates with a 50:50 Cycle Split and 2.5 sec. of End Lost Time Occurs Per Phase (2 sec. of All-Red and 0.5 sec. of Amber Interval end Lost Time)

Figure B7-1

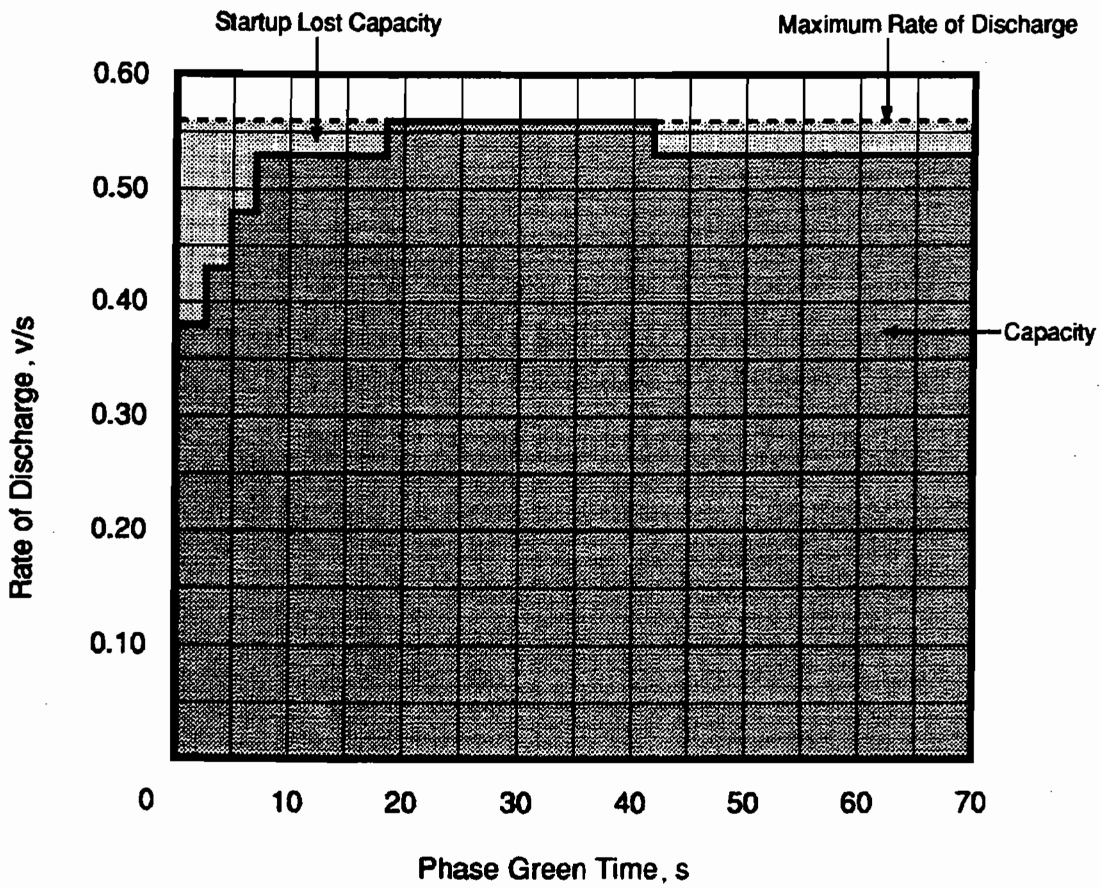
Total Lost Time at a 2 Phase Signal



Note: Assume 2 Phase Signal Operates with a 50:50 Cycle Split and 2.5 sec. of end Lost Time Occurs Per Phase (2 sec. of All-Red and 0.5 sec. of Amber Interval End Lost Time)

Figure B7-2

Capacity at a 2 Phase Signal

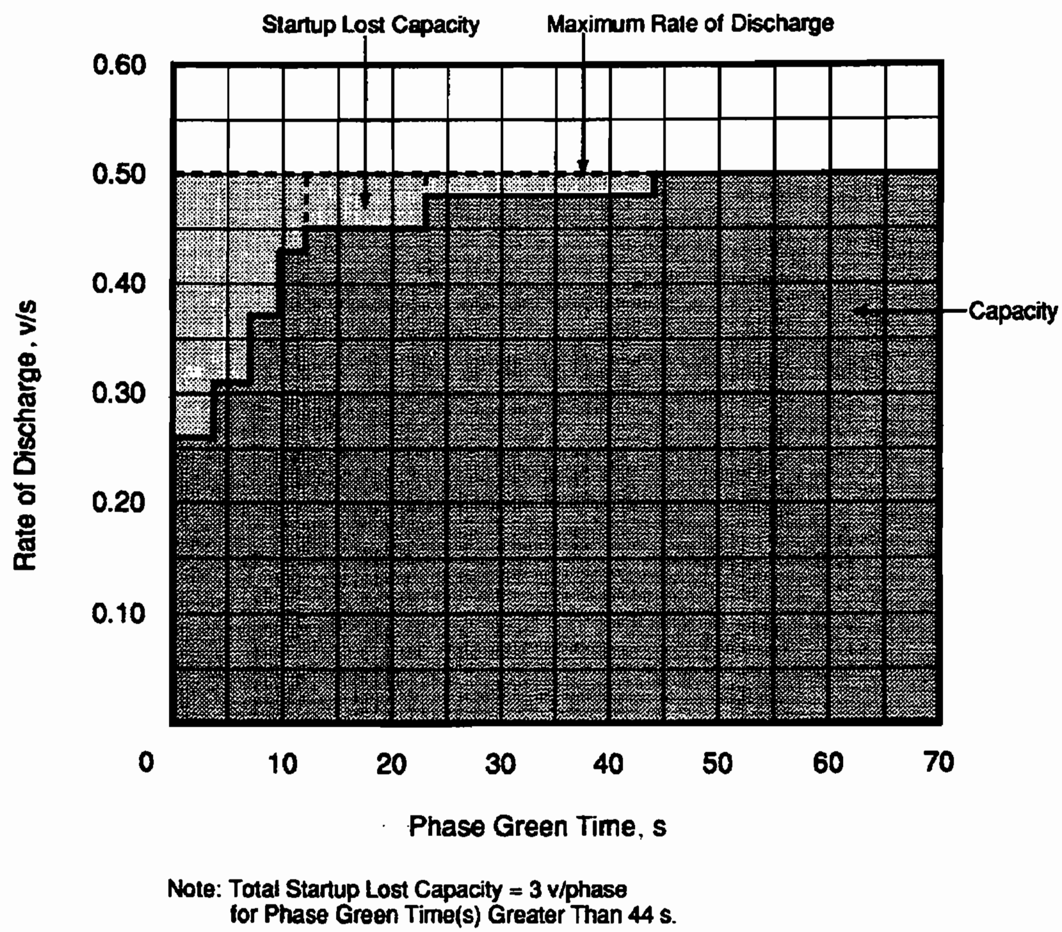


Note: Total Startup Lost Capacity = 1.3 v/phase
for Phase Green Time(s) Greater Than 18 s and Less Than 42 s.

Note: Total Startup Lost Capacity = 1.3 veh./phase
for Phase Green Time(s) Greater Than 18 sec. and Less Than 42 sec.

Figure B7-3

Rate of Discharge and Startup Lost Capacity at Urban/Commuter Intersections



Note: Total Startup Lost Capacity = 3 veh./phase
for Phase Green Time(s) Greater Than 44 sec.

Figure B7-4

Rate of Discharge and Startup Lost Capacity at Rural Intersections

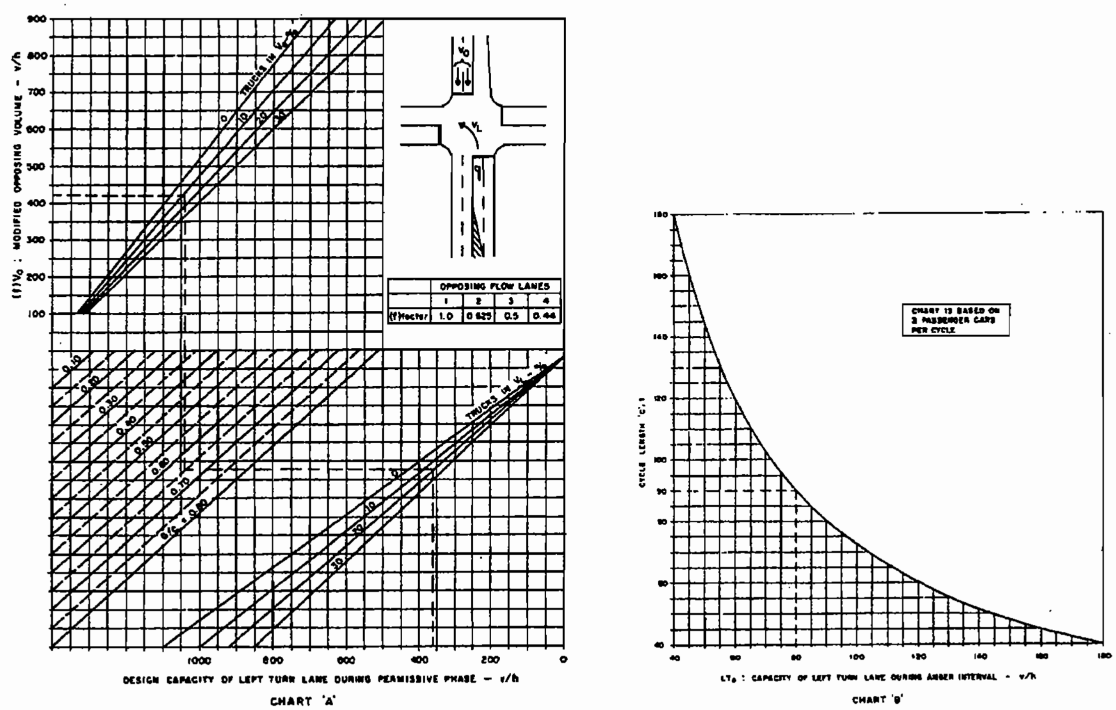
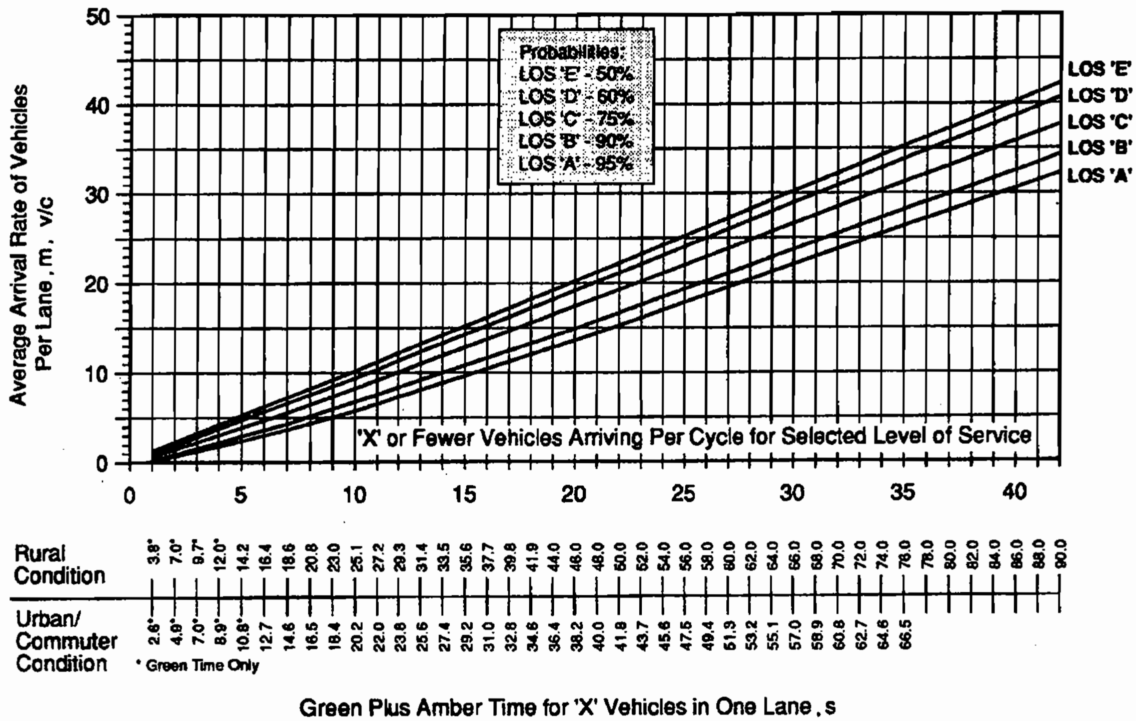


Figure B7-5

Design Capacity of Left Turn Lanes



Note: Green Plus Amber Time for "X" Vehicles in One Lane (sec.)

Figure B7-6

Green Plus Amber Times for Varying Average Arrival Rates and Levels of Service

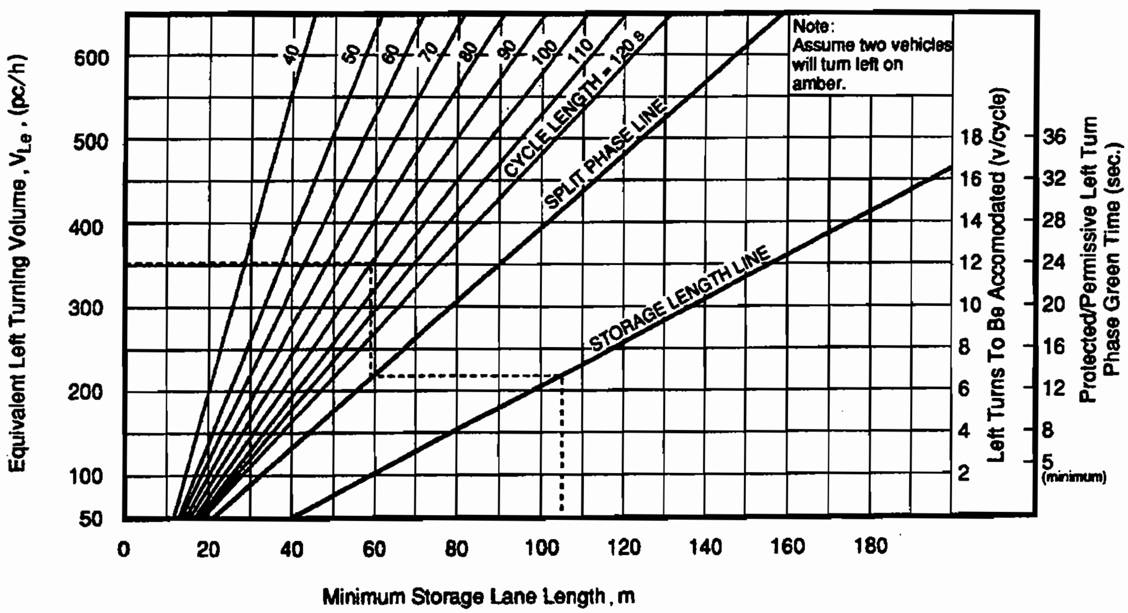


Figure B7-7

Length of Left Turn Storage Lanes and Protected / Permissive Left Turn Timing

B.8 URBAN AND SUBURBAN ARTERIALS

Urban and suburban arterials are defined as streets that have a signal spacing of not more than 3 km and whose total intersection traffic volume consists of not more than 20% turning vehicles. The primary function of these streets is to serve the through traffic while the secondary function is to provide access to adjacent development. Development can be very dense, causing friction to the traffic on the roadway and reducing the driver's speed.

In the system of urban highway transportation facilities, arterial streets are between collector and downtown streets on one side and multi-lane suburban highways and rural roads on the other. Urban and suburban arterials include multi-lane divided arterials, multi-lane undivided arterials, two-lane two-way arterials and one-way arterials.

The procedure can be used in the planning, design and operational analysis of arterials to evaluate the level of service (LOS) on an existing or proposed facility. The effect of signal spacing, arterial class (as defined here), and traffic on the LOS can be investigated. This procedure however, does not deal with service flow calculations because they are governed by the signalized intersections, which are addressed by the procedures in Section B.7.

Instead, LOS is based on the average travel speed of vehicles using the arterial.

In some cases, if the signalized intersections are operating at a very good LOS and the arterial road is designed to at least a similar standard, it may not be necessary to assess the arterial level of service. On the other hand, factors such as a lower level of design between intersections, midblock delays and large numbers of midblock entrances require an arterial analysis.

Before the arterial analysis can be done, it is necessary to analyze the signalized intersections in order to determine either the LOS or the stopped delay for the lane group containing the through traffic on the arterial. This information is used in determining the average travel speed of the arterial.

B.8.1 Factors Affecting Arterial Flow

Vehicle operation on arterials is influenced by three main factors:

- the arterial environment;
- the interaction between vehicles, and
- the effect of traffic signals.

B.8.1.1 Arterial Environment

This factor includes the following geometric characteristics of the facility and adjacent land uses:

- number of lanes, and their width
- type of median
- access point frequency
- interval between signalized intersections
- parking
- posted speed limit
- level of pedestrian activity
- urban population

The arterial environment influences the driver's perception of safe speed. If the effect of all other factors is considered to be minimal, the arterial environment restricts the driver's desired speed. In this section, the average desired speed is referred to as the free flow speed.

B.8.1.2 Vehicle Interaction

Interaction between vehicles affects the operation of vehicles at intersections and to a lesser degree between signals. Vehicle interaction is measured by the proportion of trucks, buses and turning movements in the traffic stream. In the presence of other vehicles, a driver can rarely travel at the desired speed, due to differences in desired speeds or the fact that downstream vehicles stopped at traffic signals have not reached the driver's desired speed. The driver's running speed, therefore, is normally lower than the desired speed.

In special cases, there may be unusual midblock delays or stoppages due to factors such as pedestrian crossovers or high volumes of turning traffic. These delays are recorded separately for inclusion in the analysis.

B.8.1.3 Traffic Signals

Traffic signals constrain the urban arterial capacity and lower the quality of traffic flow. Travel speed, which takes into account time lost due to intersection effects, including stops and all associated approach delay over an arterial segment is generally lower than running speed. Similarly, average travel speed of all vehicles on the segment is lower than average running speed.

B.8.2 Arterial Level of Service

Arterial level of service (LOS) is based on the average travel speed for the segment or entire arterial being considered. The average travel speed is calculated from the running time on the arterial segment(s) and the intersection approach delay.

Levels of service from A to F for urban and suburban arterials are described as follows.

Level-of-service A describes primarily free flow-operations at average travel speeds usually about 90% of the free flow speed for the arterial class. Vehicles are completely unimpeded in their ability to manoeuvre within the traffic stream. Delay at signalized intersections is minimal.

Level-of-service B represents reasonably unimpeded operations at average travel speeds usually about 70% of the free flow speed for the arterial class. The ability to manoeuvre within the traffic stream is only slightly restricted and delaysare only occasional. Drivers are not subjected to appreciable tension.

Level-of-service C represents stable operations. However, ability to manoeuvre and change lanes in mid-block locations is more restricted than in LOS B, and longer queues and/or adverse signal coordination may contribute to lower average travel speeds of about 50% of the average free flow speed for the arterial class. Drivers experience appreciable tension while driving.

Level-of-service D borders on a range on which small increases in flow may cause substantial increases in approach delay and, hence, decreases in arterial speed. This may be due to adverse signal progression, inappropriate signal timing, high volumes, or a combination of these. Average travel speeds are about 40% of free flow speed.

Level-of-service E is characterized by significant approach delays and average travel speeds of 30% of the free flow speed or lower. Such operations are caused by some combination of adverse progression, frequent signals, extensive queuing at critical intersections, and inappropriate signal timing.

Level-of-service F describes arterial flow at speeds below 25% to 30% of the free flow speed. Intersection congestion is likely at critical signalized locations, with long approach delays resulting. Adverse progression is frequently a contributor to this condition.

Table B8-1 shows free flow speed and average travel speed for levels of service from A to F. Arterial classes I, II and III are defined in Section B8.3.

Table B8-1

ARTERIAL LEVELS OF SERVICE

Arterial Class	**I**	**II**	**III**
Range of Free Flow Speeds (km/h)			
	55 to 75	50 to 55	40 to 55
Typical Free Flow Speed (km/h)			
	65	53	43
Level of Service Average Travel Speed (km/h)			
A	≥55	≥50	≥40
B	≥45	≥39	≥31
C	≥35	≥29	≥21
D	≥27	≥23	≥14
E	≥21	≥16	≥11
F	≥21	≥16	≥11

B.8.3 Methodology for Arterial Evaluation

This methodology allows for the determination of arterial level of service either with or without the use of field data. Field evaluation is a more accurate assessment of the arterial's operation, however, if field data are not available, data from a similar facility may be used. This methodology has seven steps and if field data are available, steps 4 through 6 can be superseded by field measurements of average travel speed collected by means of time and delay studies.

1. Establish the location and length of arterial to be considered.

2. Determine the arterial class, using the classification system in this section in conjunction with the measurement of free flow speed.

3. Divide the arterial into sections for the purpose of the evaluation, where each section contains one or more arterial segments.

4. Calculate the arterial running time for each segment, and the total for each section.

5. Calculate the approach delay at each intersection, taking into account:

   a) Intersection parameters for the through movement LOS or the average random intersection stopped delay per vehicle, seconds

   b) Quality of the signal progression.

   c) Relation between approach delay and stopped delay.

6. Calculate average travel speed:

   a) by section (to prepare a speed profile)
   b) over the entire facility.

7. Determine the level of service using Table B9-1.

On two-way arterials, the methodology is applied separately in each direction.

Step 1 - Establish Arterial to be Considered

The arterial location and length is precisely identified and all relevant physical, signalization and traffic data is collected.

Step 2 - Determine the Arterial Class and Free Flow Speed

For this purpose three basic arterial classes are identified and within each class there is a range of free flow speeds that are considered. The arterial class is based on its function and design.

The arterial is first classified by function, and then by design.

Functional Category

A principle arterial serves major through movements between important centres of activities in a metropolitan area and a substantial portion of trips entering and leaving the area. It also connects freeways with major traffic generators. In small cities (under 50 000), its importance is derived from the service provided to traffic passing through the urban area. Service to adjacent land is subordinate to the function of moving through traffic.

A minor arterial is a facility that connects and augments the principal arterial system. Although its main function is traffic mobility, it places more emphases on land access than the principal arterial does. Minor arterials serve trips of moderate length and distribute travel to smaller geographical areas than those served by the principal arterial.

Design Category

Suburban design represents an arterial with partial to almost full control of access with separate left-turn lanes and no parking. It may be multi-lane divided or undivided, or two-lane with shoulders. Signals are normally spaced for good progressive movement at 1 to 2.5 signals/km. Roadside development is of low density and the speed limits are usually 60 km/h to 70 km/h.

Intermediate design represents an arterial with partial control of access. It may be a multi-lane divided or an undivided one-way or a two-lane facility. It may have some separate or continuous left-turn lanes and some portions with parking permitted. It has a higher density of roadside development than the typical suburban design. It usually has 2.5 to 5.0 signals/km, and speed limits are normally 50 km/h to 60 km/h.

Urban Design represents an arterial with little or no control of access from driveways. It is an undivided one-way or two-way facility with two or more lanes. Parking is usually permitted. Generally, there are no separate left-turn lanes and some pedestrian interference is present. It commonly has 5 to 8 signals/km. Roadside development is dense with residential and/or commercial strip development. Speed limits range from 40 to 60 km/h.

Table B9-2 is used as an aid in the determination of the functional and design categories, in addition to the above definitions. Once the functional and design categories are established, the arterial class is established by referring to Table B9-3.

As a practical matter, there is some ambiguity in determining categories. The measurement or estimation of the free flow speed is a useful aid in this determination, since each arterial class has acharacteristic range of free flow speeds. The range of free flow speeds for each class are given in Table B9-1.

Free flow speed alone cannot be used to determine the arterial class, but can be used as an effective check on classification.

Step 3 - Divide the Arterial Into Sections

A segment is a length of arterial between successive signalized intersections, and is the basic unit of the arterial. The signalized intersection at the downstream end of the segment for the direction of travel being considered is included in the segment. Where two or more adjacent segments are similar in arterial class, length, speed limit and land use, they are treated as a single section. Segment lengths that vary by more than 20% are treated individually.

Step 4 - Calculate the Arterial Running Time

The length of time a vehicle spends on a section is determined by the arterial running time and the intersection approach delay. The segment running time is the travel time on the segment.

To find this the following information is required

- the arterial class
- the segment length, km
- the free flow speed, km/h

The segment running time per kilometre may be found from Table B9-4. This is then multiplied by the length of the segment to give the segment running time in seconds.

To determine the running time for a section; the average length of all segments within the section is used in Table B9-4 to give the running time per kilometre for the section. This is then multiplied by the section length to give the section running time.

Table B8-2

GUIDE TO ARTERIAL CLASSIFICATION

CRITERIA	FUNCTIONAL CATEGORY		
	PRINCIPAL ARTERIALS		MINOR ARTERIALS
Mobility function	Very important		Important
Access function	Very minor		Substantial
Points connected	Freeways, important activity centres, major traffic generators		Principal arterials
Predominant trips served	Relatively long trips between above points and through trips entering, leaving, and going through the city		Trips of moderate lengths within relatively small geographical areas
	DESIGN CATEGORY		
CRITICAL	SUBURBAN DESIGN	INTERMEDIATE	URBAN DESIGN
Control of access	PArtial to almost full	Partial	Little or no control
Arteria; type	Multi-lane divided; undivided or two-lane with shoulders	Multi-lane divided or undivided; one-way; two-lane	Undivided one-way; two-way, two or more lanes
Parking	No parking	Some parking	Parking permitted
Separate left-turn lanes	Yes	Some	No
Signals per kilometre	1 to 2.5	2.5 to 5.0	5.0 to 7.5
Speed limit, km/h	60 to 70	50 to 60	40 to 50
Pedestrian interference	None	None	SOme
Roadside development	Low density	Moderate	High density

Table B8-3

ARTERIAL CLASSES BY FUNCTION AND DESIGN CATEGORY

	Functional Category	
Design Category	**Principle Arterial**	**Minor Arterial**
Typical Suburban Design and Control	I	II
Intermediate Design	II	III
Typical Urban Design	III	III

Table B8-4

SEGMENT RUNNING TIME

Arterial Class	**I**			**II**		**III**		
Free Flow Speed, km/h	75	65	55	55	50	55	50-45	40
Average Segment Length, km				**Running Time, s/km**				
0.1							136	158
0.2				87	92	95	103	125
0.3	69	72	79	81	85	83	88	105
0.4	65	68	74	75	79	76	82	95
0.5	61	63	68					
0.6	59	61	65					
0.8	55	58	64					
1.0	54	57	64					
1.2	52	57	64					
1.4	51	56	64					
1.6	50	56	64					

Step 5 - Calculate the Intersection Approach Delay

The delay at each individual intersection is required to calculate the arterial or section speed. This may be found using the following procedure.

If random intersection delay is not already available from the signalized intersection analysis, it can be calculated as follows:

1) Refer to the signalized intersection analysis for the through movement at the signalized intersection at the downstream end of the segment. Obtain the assigned green plus amber time and the average arrival rate, m.

2) using Table B7-4 (for a rural intersection) or Table B7-5 (for an urban/commuter intersection) enter the Table at the row with green plus amber time closest to but not less than the assigned green plus amber time.

3) determine the LOS by matching the analysis value of m with the range of values in the 5 LOS columns for the selected row. If a good match is not obtained, apply the most appropriate of the following criteria:

· if m is less than the minimum value in the row for LOS A, the LOS = A

· if m falls between the ranges for two LOSs, then LOS = the poorer of the two levels, where B is poorer than A, etc.

· if m falls into the area of two overlapping ranges of LOS, then LOS = the better of the two LOSs

· if m is greater than the maximum value in the row for LOS E, then LOS = E.

4) by inspection of where m falls within the range of values for the selected LOS, estimate the location with the range of LOS as one of the following three categories:

- toward the better end of the LOS span
- near the midpoint
- toward the poorer end of the LOS span

5) using Table B8-5 select the range of random intersection stopped delay per vehicle for the LOS selected in 3) above. Select a value of d from this range based on 4) above. Note that the lower values of stopped delay are associated with the better end of the LOS span.

A progression factor, PF, is then applied to account for arrival characteristics. The arrival characteristics explain how vehicles from an upstream traffic signal arrive at the signal being considered. These are described as Types 1 to 5 as follows.

Type 1- This condition is defined as a dense platoon arriving at the intersection at the beginning of the red phase. This is the worst platoon condition.

Type 2- This condition may be a dense platoon arriving during the middle of the red phase, or a dispersed platoon arriving throughout the red phase. Better than Type 1, this is still an unfavourable platoon condition.

Type 3- This condition represents totally random arrivals. This occurs when arrivals are widely dispersed throughout the red and green phases, and/or where the approach is totally uncoordinated with other signals - either because it is an isolated location or because the nearby signals operate on different cycle lengths. This is an average condition.

Type 4- This condition is defined as a dense platoon arriving during the middle of the green phase, or a dispersed platoon arriving throughout the green phase. This is a moderately favourable platoon condition.

Type 5- This condition is defined as a dense platoon arriving at the beginning of the green phase. It is the most favourable platoon condition.

Progression factors for each type of arrival are given in Table B9-6. The stopped delay per vehicle is multiplied by the Progression Factor, PF, to give the intersection approach delay. Due to time lost during acceleration and deceleration, the intersection approach delay is greater than the intersection stopped delay.

Intersection Approach Delay = 1.3 x Stopped Delay per vehicle.

If a section is being considered, Intersection Approach Delay = The sum of individual Intersection Approach Delays.

The Intersection Approach Delay is used in the next step to calculate average travel speed.

Step 6 - Calculate Average Travel Speed

Average Travel Speed is calculated on a section-by-section basis for the entire length of the arterial, using the running time determined in Step 4 and the intersection approach delay determined in Step 5.

If there are appreciable mid-block delays, such as pedestrian cross-overs or major traffic generators, an additional term for mid-block delay can be added to the denominator of the equation for averge travel speed.

Average Travel Speed =

$$\frac{\text{length} \times 3600}{(\text{Section running time}) + (\text{Intersection Approach Delay})}$$

where:

- Average travel speed is in kilometres per hour
- Length is section length in kilometres
- Running time is in seconds per kilometre
- Intersection Approach Delay is in seconds

Step 7 - Assess Level of Service

Level of service is determined from Table B9-1 using the average travel speed found in Step 5.

There is a distinct set of arterial level-of-service values established for each arterial class. These are based on the differing expectations of drivers and judged to have for the different classes of arterials.

In defining the levels of service, both the free flow speed of the class and the intersection LOS definitions are taken into account. In general, the arterial levels of service are based on the smooth and efficient movement of the through traffic along an entire arterial. Therefore, it is necessary to expect less delay per segment than the corresponding intersection level of service.

Table B9-1 gives the arterial level-of-service definitions for each of the three arterial classes. The level-of-service definitions vary with the arterial class: the lesser the arterial class (i.e. the higher the class number), the lower the driver's expectation while driving on that facility, and the lower is the speed associated with a given level of service. Thus, a Class III arterial provides LOS B at a lower speed than does a Class I arterial.

If reconstruction results in upgrading a facility from Class II to Class I, it is possible that the LOS will not change despite average speed and other improvements, because expectations would be higher.

The concept of an overall arterial level of service is generally only meaningful when all segments on the arterial are of the same class. If there are different arterial classes represented, the LOS criteria are different.

Table B8-5

LEVEL-OF-SERVICE CRITERIA FOR SIGNALIZED INTERSECTIONS

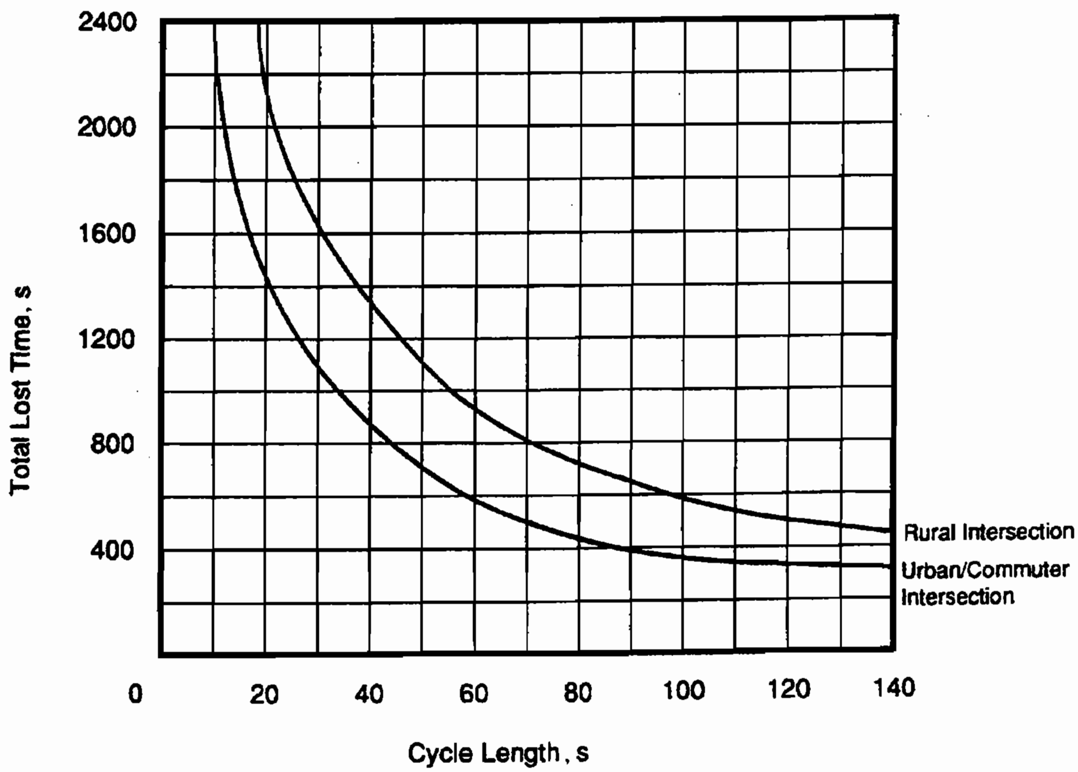
Level of Service	Stopped Delay Per Vehicle (s)
A	≤5.0
B	5.1 to 15.0
C	15.1 to 25.0
D	25.1 to 40.0
E	40.1 to 60.0
F	> 60.0

Table B8-6

PROGRESSION FACTOR

Type of Signal	Lane Group Types	Level of Service	v/c Ratio X	Arrival Type				
				1	2	3	4	5
Pretime	Through Right Turns	A	0.66	1.68	1.29	1.00	0.74	0.58
		B	0.72	1.59	1.25	1.00	0.78	0.62
		C	0.87	1.45	1.20	1.00	0.84	0.72
		D	0.99	1.41	1.18	1.00	0.89	0.81
		E	1.00	1.40	1.18	1.00	0.90	0.82
Actuated	Through Right Lanes	A	0.66	1.42	1.04	0.85	0.64	0.43
		B	0.72	1.33	1.02	0.85	0.67	0.46
		C	0.87	1.20	0.97	0.85	0.73	0.53
		D	0.99	1.17	0.95	0.85	0.78	0.60
		E	1.00	1.16	0.95	0.85	0.78	0.61
Semi-actuated*	Main Street, Through Right Lanes	A	0.66	1.68	1.29	1.00	0.74	0.45
		B	0.72	1.59	1.25	1.00	0.79	0.48
		C	0.87	1.45	1.20	1.00	0.84	0.57
		D	0.99	1.41	1.18	1.00	0.89	0.64
		E	1.00	1.40	1.18	1.00	0.90	0.65

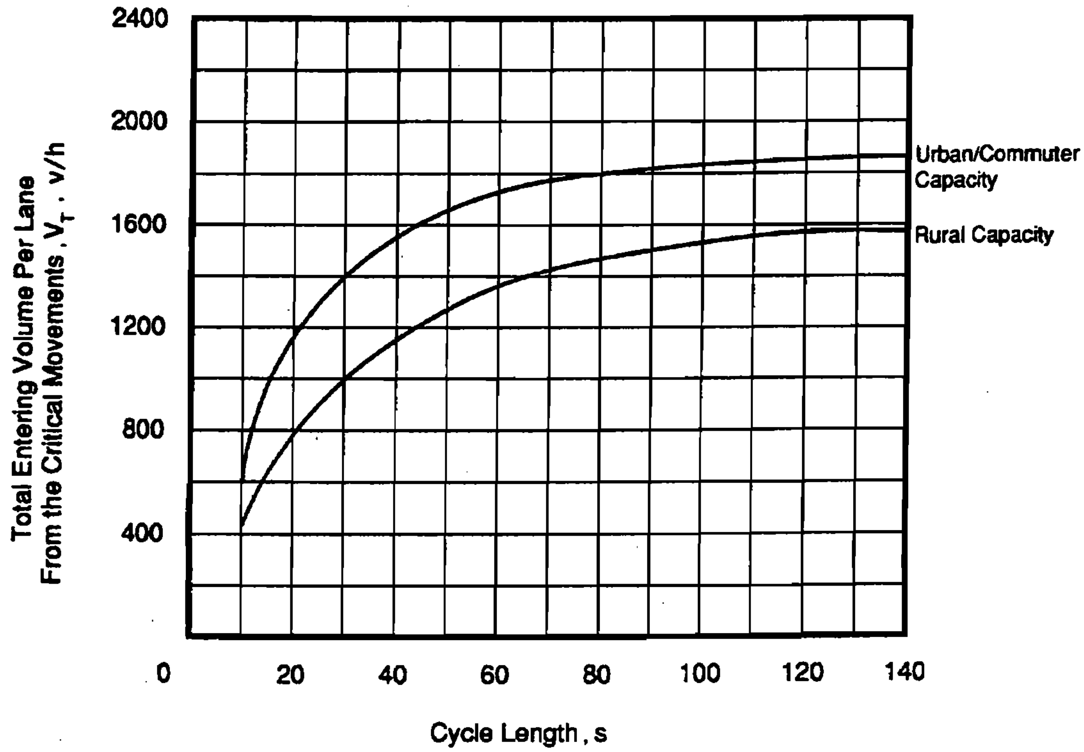
*Semi-actuated signals are typically timed to give all extra green time to the main street. This effect should be taken into account in the allocation of green times.



Note: Assume 2 Phase Signal Operates with a 50:50 Cycle Split and 2.5 sec. of End Lost Time Occurs Per Phase (2 sec. of All-Red and 0.5 sec. of Amber Interval end Lost Time)

Figure B8-1

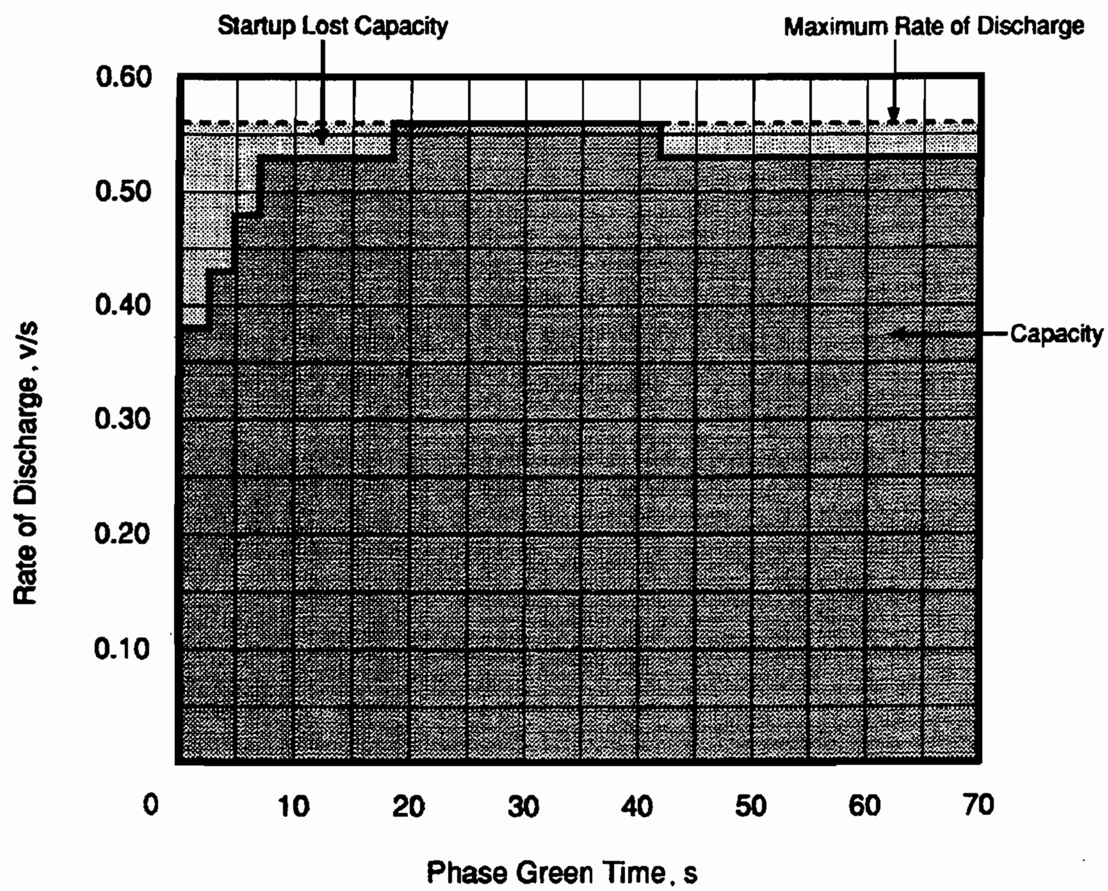
Total Lost Time at a 2 Phase Signal



Note: Assume 2 Phase Signal Operates with a 50:50 Cycle Split and 2.5 sec. of end Lost Time Occurs Per Phase (2 sec. of All-Red and 0.5 sec. of Amber Interval End Lost Time)

Figure B8-2

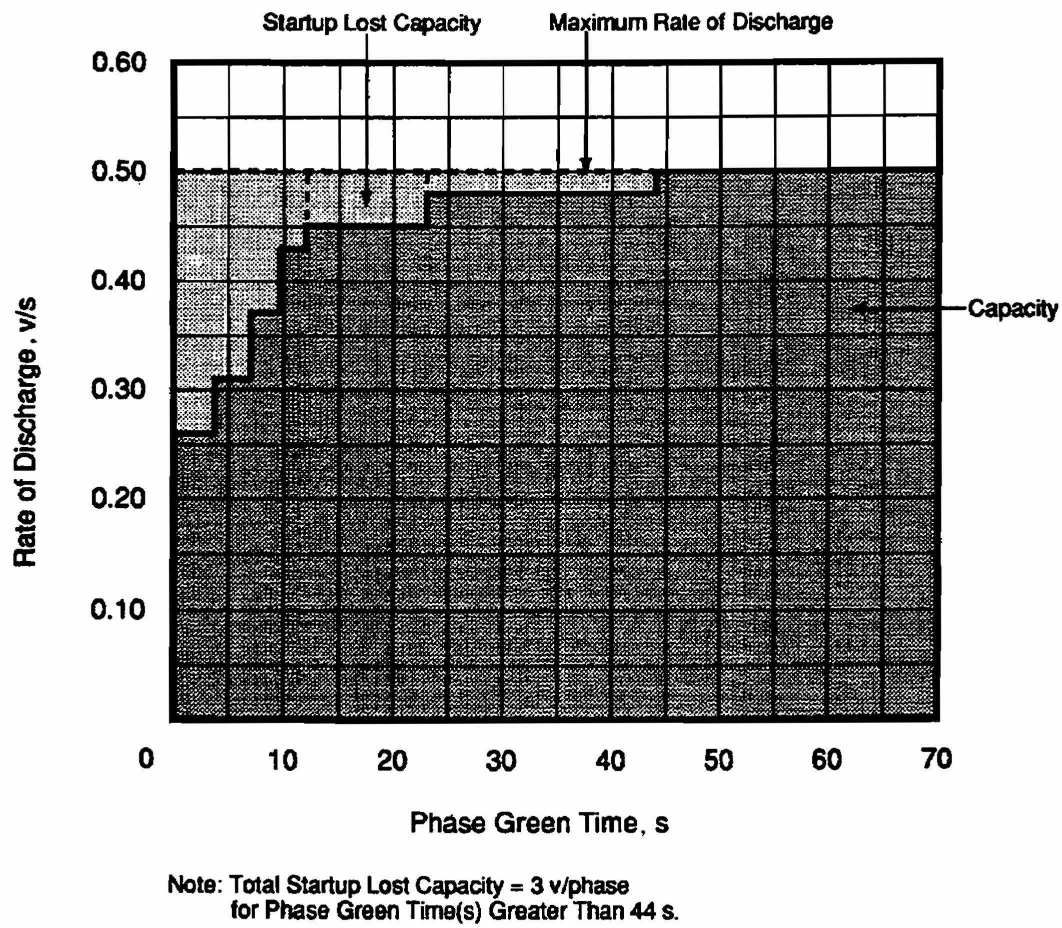
Capacity at a 2 Phase Signal



Note: Total Startup Lost Capacity = 1.3 veh./phase
for Phase Green Time(s) Greater Than 18 sec. and Less Than 42 sec.

Figure B8-3

Rate of Discharge and Startup Lost Capacity at Urban/Commuter Intersections



Note: Total Startup Lost Capacity = 3 veh./phase
for Phase Green Time(s) Greater Than 44 sec.

Figure B8-4

Rate of Discharge and Startup Lost Capacity at Rural Intersections

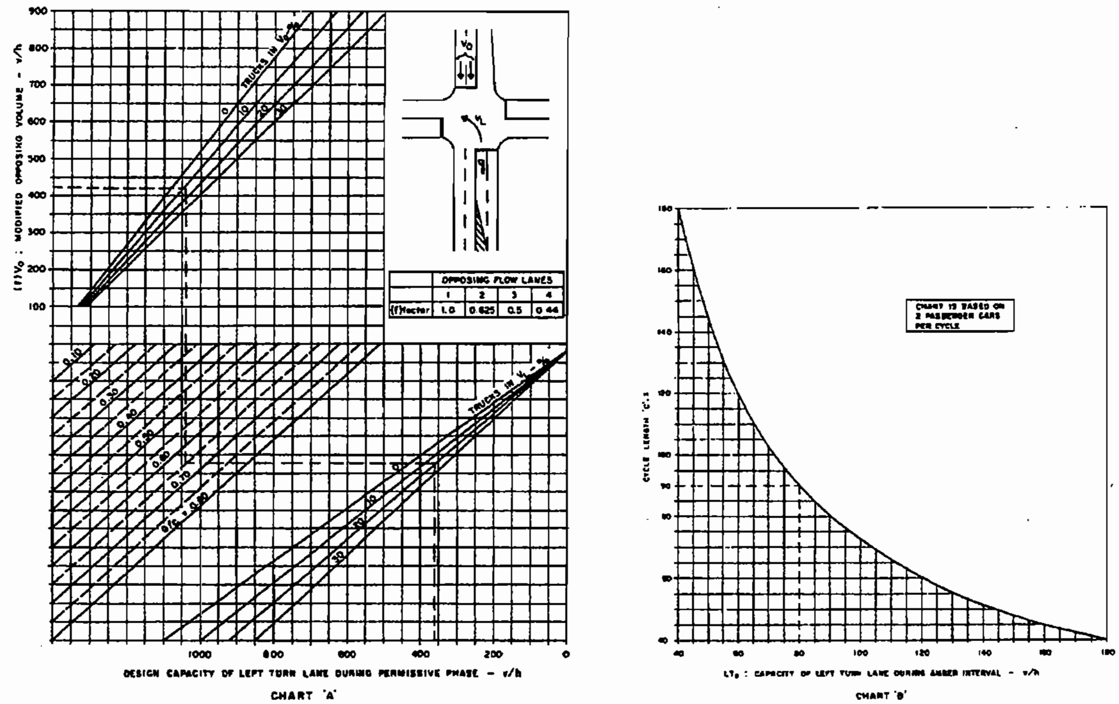
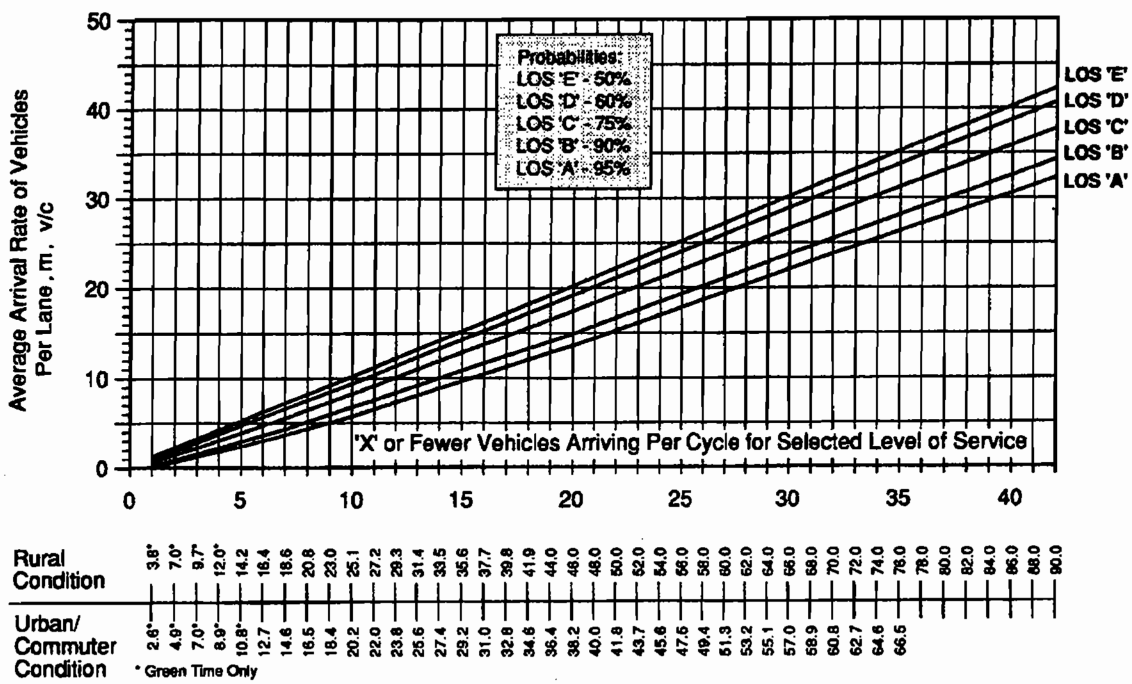


Figure B8-5

Design Capacity of Left Turn Lanes



Note: Green Plus Amber Time for "X" Vehicles in One Lane (sec.)

Figure B8-6

Green Plus Amber Times for Varying Average Arrival Rates and Levels of Service

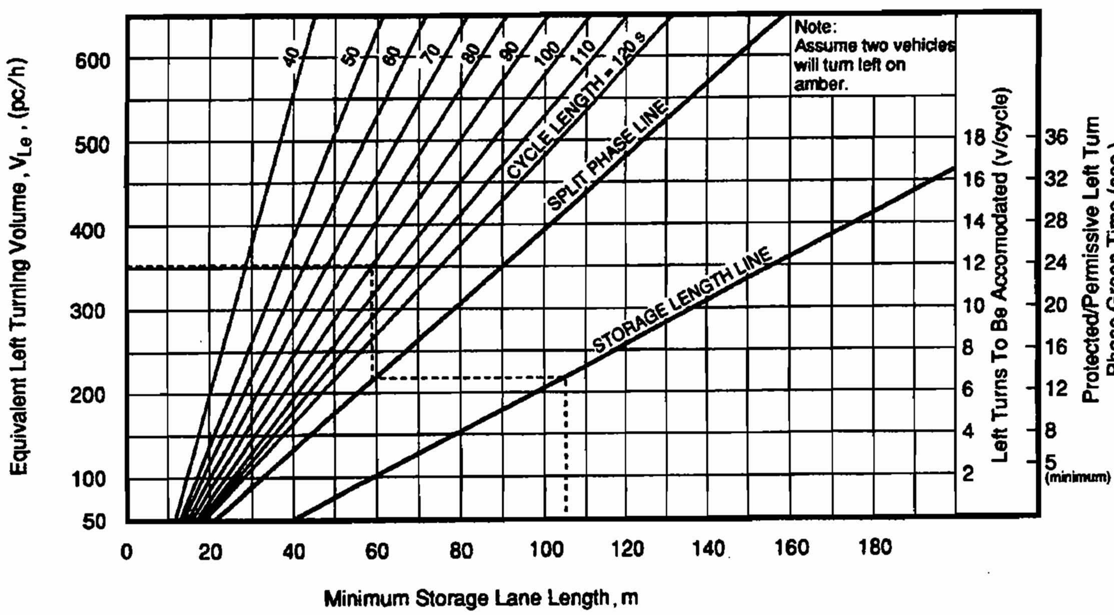


Figure B8-7

Length of Left Turn Storage Lanes and Protected / Permissive Left Turn Timing