CANADIAN CAPACITY GUIDE FOR SIGNALIZED INTERSECTIONS

The Software is InterCalc

Third Edition
February 2008

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Canadian Capacity Guide for Signalized Intersections

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Committee on the Canadian Capacity Guide for Signalized Intersections

Subject areas: Roads - interchanges and intersections. Electronic traffic controls

Traffic flow - mathematical models.

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Connection to InterCalc Software

CITE is committed to maintaining the Guide as a storehouse for Canadian traffic engineering knowledge related to signalized intersections. It is CITE's intention to promote the Guide methodology as best practice, within Canada and beyond.

The previous editions of the Guide left the question of software to replicate its procedures open to the user community. CITE has now recognized that for the Guide to continue as a living document, software which replicates its procedures is essential. Under a formal arrangement with CITE, the BA Consulting Group of Toronto has developed (at its own cost) a software that replicates the procedures of the Guide. That InterCalc software is endorsed by CITE. InterCalc ordering and technical information can be found at www.intercalc.ca.

The software also includes processes for analyzing unsignalized intersections. The Guide does not address unsignalized intersections. This unsignalized software module is based on the most recent Highway Capacity Manual, with a number of enhancements over the HCM method. These are intended to address multi-lane approaches, (including two through lanes plus exclusive left turn lanes) and to correct minor errors in the HCM method.

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David Checkel from the Department of Mechanical Engineering at the University of Alberta developed the fuel consumption and emission units, and tested the evaluation models.

Much of the research that forms the backbone of this Guide has been carried out at the University of Alberta and supported by the Natural Sciences and Engineering Research Council of Canada. Several techniques included here have been developed for the 2000 updates of the US Highway Capacity Manual, and the contributions of the members of the Transportation Research Board Committee A3A10 on Highway Capacity and Quality of Service are gratefully acknowledged. We are also indebted to our Australian, British and German colleagues who pioneered many of the techniques on which we have based our approach.

This Third Edition of the Canadian Capacity Guide for Signalized Intersections is an updated, enhanced and significantly expanded version of this important traffic engineering document.

Canadian Capacity Guide Development Committee

In the year 2000, the Canadian Institute of Transportation Engineers (CITE) committed to updating and enhancing the Guide, and to supporting the development of software that replicates its procedures. The core group which has guided this work has included:

Committee Members

Tim Arnott, ... BA Consulting Group
Jim Gough, ... MMM Group
Robert McBride, ... BA Consulting Group
Chris Middlebro', ... BA Consulting Group
Dave Richardson, ... MMM Group
Paul Sarjeant, ... BA Consulting Group

Joint CITE-TOMSC Advisory Committee

CITE and the Traffic Operations and Management Standing Committee of the Transportation Association of Canada have set up a joint Advisory Committee to support the ongoing development of the Guide, its methodologies and its databases. The members of the Committee are:

TOMSC

Mike Skene, ... Boulevard Transportation Group (TOMSC Committee Chair)
Suzanne Beale, ... Town of Whitby
Robert Kahle, ... Ville de Montreal
Troy McLeod, ... City of Calgary
Hart Solomon, ... City of Hamilton
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CITE

Jim Gough, ... MMM Group (CITE Committee Chair)
Jamie Copeland, ... Delphi-MRC
Jean-Philippe Desmarais .... Roche Ltée

The interest and support of many other Canadian transportation professionals were an important stimulus to the Committee.

Disclaimer

The procedures contained in this Guide provide information that assists in planning and design decisions but they are not a substitute for professional judgment regarding societal, environmental, behavioural, legal and other factors. Specific local conditions, requirements and constraints must be considered, even if they are not explicitly mentioned in this document.

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CHAPTER

1

OBJECTIVES AND CONTENTS OF THE GUIDE

This Guide has been based on the current experience of practicing traffic engineers, transportation educators and students across Canada, and a considerable body of Canadian and international research. But while the Guide has been developed in Canada, its methodology is applicable to conditions anywhere. The application of locally relevant parameters will enhance the Guide's utility.

Many cities and metropolitan areas experience traffic congestion on some portions of their transportation networks. These municipalities also suffer from constrained urban space and limited financial resources, but they share the desire to improve the quality of their environment. The analytical tools to understand specific problems require refined methods for the evaluation of alternative solutions.

Techniques included in this Guide allow the user to analyze various situations and intersection configurations. This Guide emphasizes the importance of a clear definition of the objectives of signal operation at a specific location. It also provides an understanding of the role that the intersection plays in the travel patterns, public transportation, and both motorized and non-motorized modes of transportation.

The focus of the Guide is on the movement of traffic flow units, such as cars, trucks, transit vehicles, cyclists, and pedestrians at signalized intersections. The main parameter is the time dimension that determines how efficiently the available roadway space is used by conflicting traffic streams. The allocation of time to the movement of vehicular and pedestrian traffic in lanes and crosswalks influences not only intersection capacity, but also a number of other measures that describe the quality of service provided for the users. To this end, and to provide input to investigations of possible impacts, the Guide provides both analytical and evaluation methods, and a set of up-to-date numerical parameters for Canadian conditions.

While these parameters are Canadian, the method is widely applicable. The survey procedures included in the Guide provide direction for users in any country to collect local data which can be used to obtain geographically specific results.

Using the Guide, it is possible to assess a variety of solutions by application of a set of practical evaluation criteria. The evaluation criteria, or measures of effectiveness, provide the user with a comprehensive account of intersection operation. Two of the key measures of effectiveness are total person delay and delay to pedestrians. These criteria are essential as the prerequisites for an equitable treatment of all modes of transportation, especially public transit. Other performance measures relate the Guide to environmental, economic and safety analyses, and serve as vital information for transportation demand modelling.

Delay and the ratio of volume to capacity are two key parameters widely used in the profession to assess the performance of an intersection. The Guide focuses on the ratio of volume to capacity as a rational measure of how well the intersection is accommodating demand, but it is acknowledged that delay is also widely used (for example, in the Highway Capacity Manual). Whether one parameter or the other is the most relevant is the subject of ongoing debate in the profession. It is advisable to consider both parameters in the assessment of an intersection, at the level of the individual movement, the approach and the intersection as a whole.

1.1 Objectives

The objectives of the current Guide are as follows:

- to update and expand the Guide with respect to current practice;
- to consolidate the available Canadian information and experience on planning, design, and evaluation of signalized intersections in one document;
- to contribute to information exchange among Canadian transportation and traffic engineering professionals, and to further develop an advanced national practice;
- to provide guidance for both experienced and novice practitioners;
- to assist in the education of present and future transportation professionals.

1.2 Scope

The Guide provides a set of techniques that can be applied to operational, design and planning problems at signalized intersections. The operational procedures deal with a detailed assessment of operating conditions within a relatively short time frame when all factors are known or can be reasonably estimated. The design process is used to determine specific control parameters and geometric features of an intersection that will meet desired design objectives and performance criteria. Planning techniques, often called functional design, are useful for longer range problems, assisting in the determination of the type of the facility and its basic dimensions. The basic method remains the same for all three application types, but the level of detail varies.

Wherever possible, the Guide utilizes formula-oriented techniques that can be applied both in manual calculations and computer programs, including spreadsheet tables. Although advanced simulation and other computerized techniques may prove to be superior to formula-based methods in the future, the understanding of the fundamentals contained in the Guide remains essential.

Where practical, measured input parameters and measured output performance criteria are preferable to calculated values. Correct and consistent survey methods as well as a critical assessment of the degree of precision and reliability of the survey results are essential.

The principles and components of the timing design and evaluation processes are based on the international state-of-the-art in both the research and practice for intersection control. As a consequence, a knowledgeable user will find many similarities to other international documents. Nevertheless, some individual procedures, especially with respect to saturation flow and evaluation criteria, may differ because they were developed, tested, or adjusted for specific Canadian conditions. Some methods and parameter values are a direct result of the work on this Edition, but wherever possible, the original references or sources are identified.

The Guide allows the evaluation of existing or future intersection control or geometric conditions relative to travel demand. It does not deal directly with broad systems or network issues, such as transportation demand management or congestion management. The results of the procedures included in the Guide, however, can be used as information for the evaluation of the impact of intersection control, or geometric alternatives on system aspects, such as population mobility, accessibility of various destinations or land use strategies. Although safety is an integral part of all traffic considerations, the Guide does not address this broad and complex issue explicitly. It is left to other specialized documents.

Similar to the First Edition, the new version of the Guide concentrates mostly on urban applications. Although the procedures focus on fixed-time signal operation, advice is provided for their adjustment to the design and evaluation of traffic responsive signal control, including the traffic actuated method.

1.3 Links to the U. S. Highway Capacity Manual (HCM)

Whereas this Guide deals only with signalized intersections, the Highway Capacity Manual (TRB 2000) covers a whole range of transportation facilities. Many Canadian jurisdictions rely on the HCM procedures developed for general North American conditions for freeways, multilane roadways, two-lane two-way highways, intersections without traffic signals, and pedestrian, bicycle, and transit facilities.

Signalized intersections constitute a somewhat special case. City structures, geometric design practices, and the behaviour of the users of these transportation facilities vary greatly across the continent, but show many similarities in Canada. Moreover, analytical and design methods for signalized intersections in Canada have a long tradition based in many aspects on British research and techniques (Webster 1958, Webster and Cobbe 1966). These factors, and the critical importance of signalized intersections in urban networks, resulted in the need for a specifically Canadian document.

The principles employed in Chapter 16: “Signalized Intersections” of the Highway Capacity Manual (TRB 2000), and the principles contained in this Guide have identical theoretical foundations. The documents differ in the application of these basic principles, in the measured values, and in the calibrated relationships that reflect specific conditions in both countries. The Guide establishes a link between the average overall delay used here, and the average stopped delay applied in the Highway Capacity Manual for the determination of the level of service.

1.4 The structure of the Guide

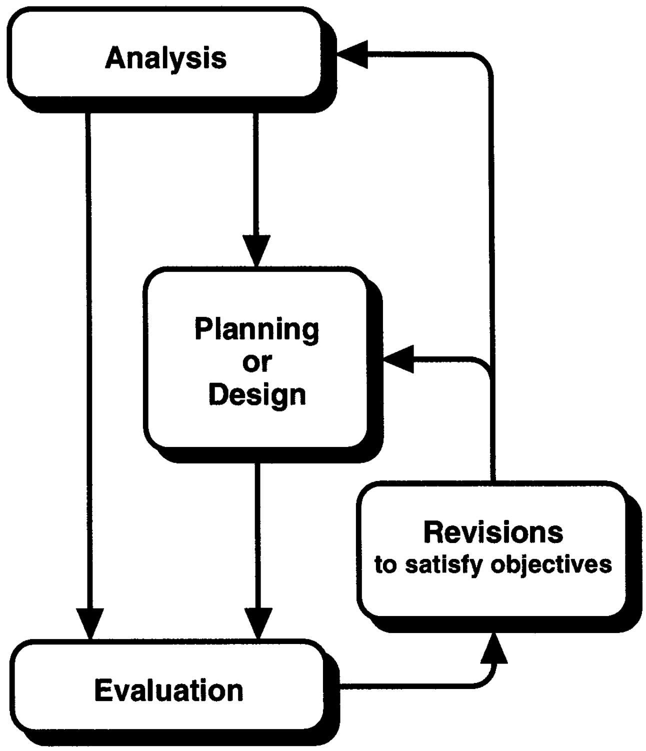


Figure 1.1 Analysis, planning and design, and evaluation of signalized intersections.

Chapters 1 through 4 **introduce and describe the methods and parameters.** *Chapter 5* **provides instructions for the survey techniques.** *Chapter 6* **illustrates the process in a number of design examples.**

In *Chapter 3*, the basic considerations and input variables are introduced in the *Analysis* Section. The *Planning and Design* Section that follows, describes the process of determining essential signal timing parameters. The subsequent *Evaluation* Section allows the user to assess intersection performance based on existing or future control parameters and geometric features.

A simplified process of analysis, planning and design, and evaluation is illustrated in Figure 1.1.

The *analytical* tasks provide basic information on demand and how it is influenced by local conditions. If the intersection exists, and the traffic flow as well as all geometric, control and other features are known, this analytical information becomes an input to *evaluation* of its operations. The software which replicates the Guide's procedures addresses the vast majority of the processes contained within it. The software has been tested by personnel experienced in the application of its techniques.

Evaluation of the operation can be summarized as follows:

- Initial saturation flow values are assigned to each movement based on the type of movement (i.e. left turn, through, right turn)
- The saturation flow values are adjusted for locally specific conditions, in terms of the type of signal operation, presence of pedestrians, etc.)
- The arrival flow of traffic is related to the adjusted saturation flow to determine the degree of utilization for each movement
- The level of service is calculated for each movement, in terms of volume to capacity ratio and/or delay

- The overall level of service is calculated based on the degree of utilization of the "critical movements" for the intersection - typically some combination of left turns and high through volumes are the critical movements, which effectively define the level of service for the intersection as a whole

The concept of an overall intersection level of service is one that is debated within the profession. Some contend that only the level of service for individual movements or approaches is meaningful and based in proven science. Others contend that an overall level of service is useful in communicating with audiences who do not have a detailed understanding of traffic engineering; in those situations, an overall level of service is regarded as a helpful shorthand to communicate how the intersection is performing, relative to accepted community standards and other intersections which the audience may be familiar with.

If the intersection is in the *planning or design* stages, the expected demand and other characteristics must first be analyzed in order to determine the necessary parameters for the evaluation of future operations. For both existing and future intersections, however, the evaluation may reveal that the operation does not meet the desired objectives. In that case, the design and operating strategy must be adjusted until the specified performance objectives and parameters are met.

1.5 Major changes since the Second Edition

The Second Edition, published in 1995 (ITE 1995), was reviewed in order to accommodate new research results, increased practical knowledge and user experience. The main principles of analysis, design and evaluation have been maintained.

The principal changes are as follows:

Format

- Text has been re-organized slightly and standardized
- Format has been changed to improve readability

Contents

- Minor corrections to formulae and text from the Second Edition
- More worked examples
- Expanded discussion of safety
- Expanded discussion of traffic responsive operation
- Expanded discussion of transit priority operation
- Update on saturation flow values
- Expanded discussion of level of service

1.6 Intersection Geometric and Control Elements

Figure 1.2 provides an illustrative definition of geometric and control elements of a signalized intersection. It should be noted that this example attempts to include as many geometric configurations as possible. Some elements, such as the offset north/south left turn lanes, are not typical, and have various benefits and disbenefits in specific situations. No endorsement of such devices, lane designations or lane alignments is intended.

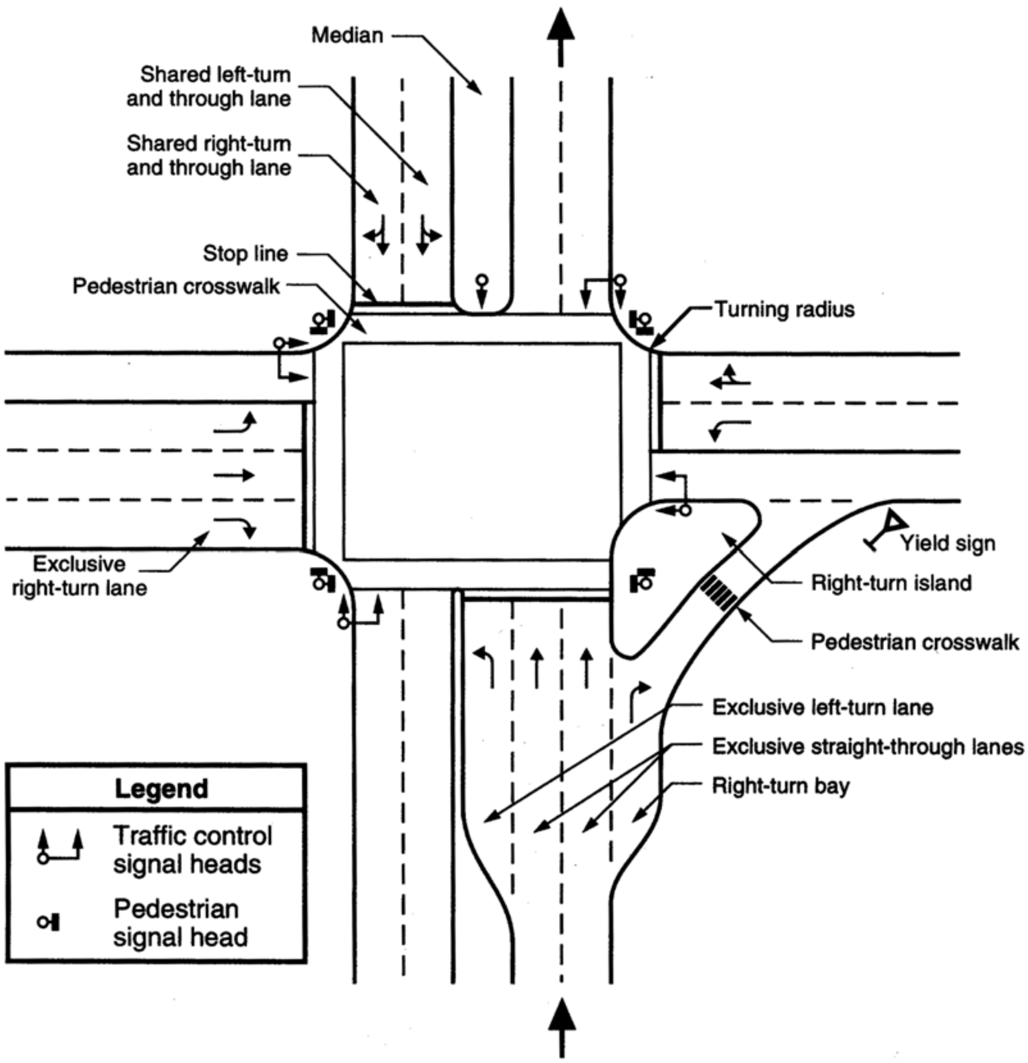


Figure 1.2 Geometric and control elements of a signalized intersection

CHAPTER

2

HOW TO USE THIS GUIDE

2.1 The process

This document describes a set of procedures and values for the analysis and evaluation of traffic operations at signalized intersections based on Canadian best practices. It also includes information regarding the planning and design of these facilities.

The Guide is not a textbook. For basic education on the subjects of traffic flow theory, traffic operations and capacity, users are referred to appropriate texts in traffic and transportation engineering. The explanations included in this document are intended to enhance understanding, not to deal comprehensively with fundamentals.

The Guide provides the background information for planning, design, and evaluation of physical features and signal timing requirements of signalized intersections. The applicability of this information must be subject to professional judgment regarding legal, societal, environmental, behavioural, and other specific local conditions, requirements, and constraints.

2.1.1 Factors involved in planning, design, and evaluation

The operation of traffic control signals at an intersection introduces interrupted flow conditions on all approaches. Traffic signal indications and the rules of the road provide for the time-shared use of the common space by vehicular and pedestrian flows entering from various approaches and departing to different directions. The following six categories of factors have a major influence on the traffic flow through an intersection:

- horizontal and vertical geometry of the intersection and adjacent roadways (geometric conditions)
- control conditions
- traffic conditions
- rules of the road
- environmental conditions
- user behaviour

Many individual factors in these categories are listed as inputs to the analytical process in this Guide. Traditional geometric or control practices and driver and pedestrian behaviour vary from region to region. The rules of the road may also be different for individual provincial or municipal jurisdictions. The user of the Guide must therefore make sure that the assumptions in the computational technique used in a specific case do not contradict established local practices or behavioural habits, and do not violate local legal requirements.

2.2 Calculation models

The six factors listed in *Section 2.1.1* must be considered in all stages of the analytical, design, planning, and evaluation process identified in the introductory section of the Guide. The techniques presented in the Guide include some of these factors or their combinations, and can be applied in formulas, tables and worksheets. The software which replicates the Guide's procedures addresses the vast majority of the processes contained within it. The software has been tested by personnel experienced in the application of its techniques.

The user is advised to consider other types of representation of local conditions that may be appropriate for the given problem. Data obtained from local surveys can be used to describe specific conditions and is, in fact, a model of the conditions measured. Computer models in the form of iterative software are well suited for design problems. Models that track vehicles and pedestrians in time and space in a computer simulation may provide the best basis for the evaluation of intersection performance under varying traffic and other conditions, since they may include many complex interactions among the factors involved.

It is the analyst's responsibility to select the most appropriate model or technique for the problem at hand.

2.3 Precision and accuracy

The degree of precision applied to individual tasks in the process depends on the intended use of the results and the accuracy of the input values. Keep in mind that traffic is subject to random fluctuations as well as variations resulting from influences that can not be included in analytical formulations. Input variables are rarely known with absolute certainty.

Measured output variables vary. Many of the relationships included in the Guide therefore yield averages or typical values. Several evaluation criteria in *Section 4.6* attempt to capture the probabilistic nature of the traffic process.

2.4 Steps in the process

Analysis, planning, design, and evaluation are concerned with securing efficient and safe conditions for existing or expected users of the intersection. *Figure 2.1* gives an overview of the activities involved.

Definition of objectives

Specific objectives and constraints may vary from facility to facility, and should be identified at the outset of the process shown in *Figure 2.1*. The objectives must be based on strategic goals of the broader system or network. They should be clearly stated and expressed in measurable operational criteria. Examples of these objectives may include: minimization of average overall vehicle delay; equitable allocation of vehicle or person delay to individual intersection approaches or lanes; minimization of vehicle delay for some direction and incorporation of a delay penalty for other directions to control “shortcutting”; maximization of vehicle capacity; control of queues; minimization of gridlock risk; minimization of delay to pedestrians, etc.

Note that while intersection capacity for a given set of factors is relatively easy to determine for vehicular movements, person capacity depends on prevailing average vehicle occupancies of individual vehicle types. The use of maximum occupancies of passenger cars or buses does not provide a reliable estimate of person capacity, and is not an adequate basis for planning, design, and evaluation.

Analysis stage

This stage involves the investigation of specific intersection conditions and the determination of relevant evaluation, design or planning parameters. Vehicle flows and saturation flows are the cornerstones of the whole process and must be reliably determined. Existing signal timings and constraints (in the case of analysis of existing conditions,) or preliminary timing considerations (in the case of a future change in conditions,) are also used as inputs to the planning, design and evaluation procedures.

The analyst should consider an appropriate complexity level for a given task. The Guide recognizes this requirement in the simplified input for the planning process. A redesign of an existing signal operation, on the other hand, requires detailed input, if necessary by direct measurement of flows and saturation flows. A set of detailed procedures to estimate lane-by-lane flows and saturation flows is provided for those instances where the measurements are not possible because the intersection does not yet exist, or the surveys would be too time consuming or costly.

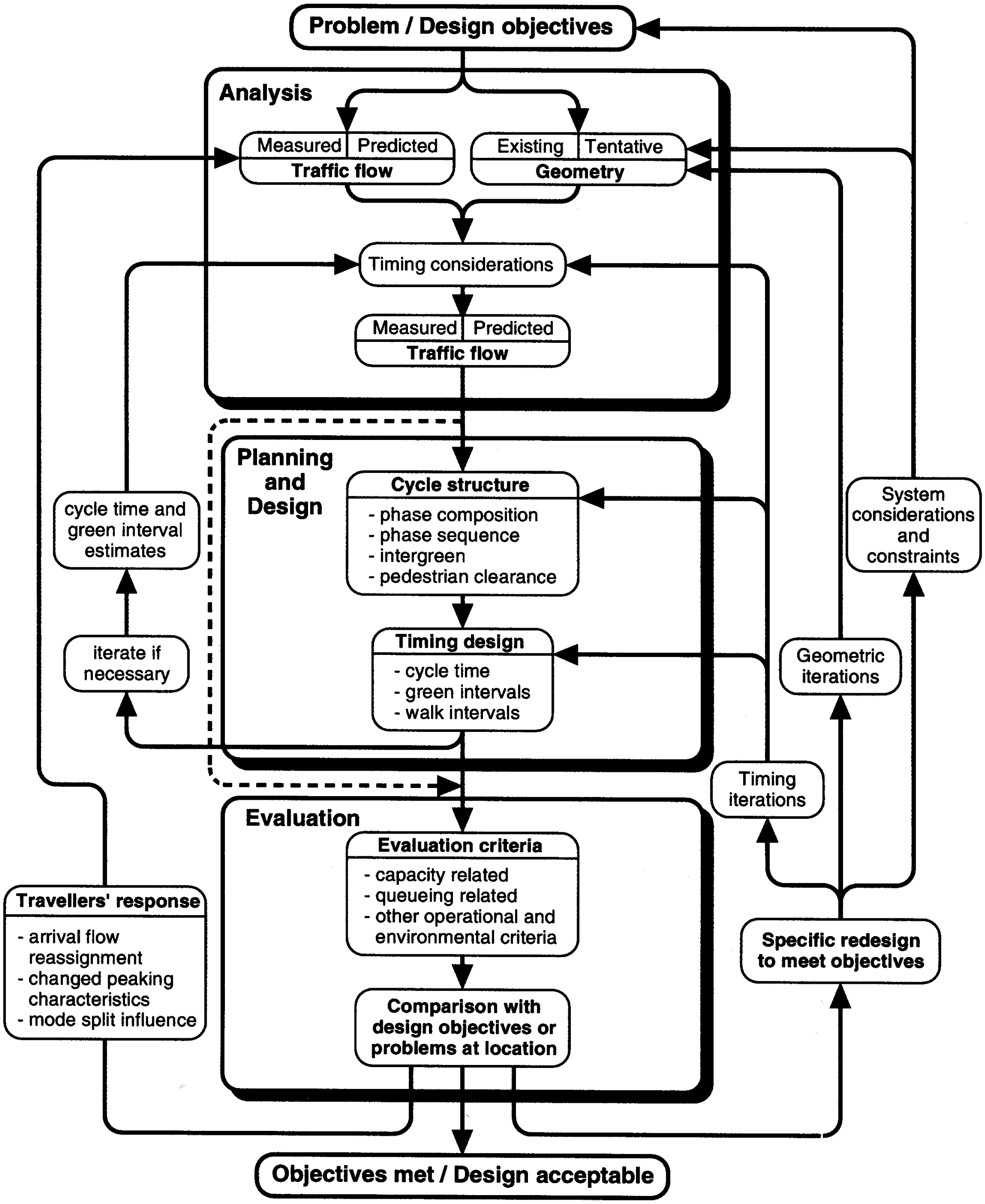


Figure 2.1 Elements of the analysis, planning, design, and evaluation of signalized intersections

Planning and design stages

The term "planning" is used throughout the Guide to designate what is often termed "functional planning" or "functional design", in which the focus is on the definition of essential geometric features for the future. The "design" process is then concerned with detailed operational parameters, such as the structure of the cycle, cycle time, and individual signal intervals. In many instances, the design also involves detailed geometric features, such as the dimensions of approach lanes and their assignment to intersection directions, identification of queueing space requirements, and pedestrian refuges.

With the exception of very simple cases, the design process is usually iterative, as indicated in the right-hand feedback loop in *Figure 2.1*. The results of the initial timing design are evaluated and tested against specific objectives and constraints of the desired operational performance. After the evaluation, it is frequently necessary to reconsider some of the design features and to adjust the initial timing design. The most common adjustments involve the composition of the phases and the cycle structure, (i.e. the assignment of individual movements to phases and lanes,) and the sequence of phases. These adjustments may, in turn, result in a different cycle time and signal intervals.

If these timing adjustments fail to meet the desired objectives, modifications to the geometric features may be appropriate. The changes may be as simple as changing the allocation of individual intersection movements to lanes, or as complex or expensive as the addition or removal of lanes, turning bays, or channelization islands.

Existing intersections operating under geometric constraints are usually restricted in the range of feasible modifications. Planned intersections that will be built and made operational in the future may allow more freedom to vary the input variables. On the other hand, existing intersections usually provide the designer with a known set of traffic flow and geometric conditions, rather than the uncertainty related to projected traffic and other conditions for facilities not yet built.

Similar to many other engineering tasks, some of the design output values may be needed within the design process. The most frequently required values of this kind are the cycle time and green intervals. Since they are frequently not yet known at the initial design stage, they must be estimated. If their output values differ substantially from the estimates, it may be necessary to reiterate the appropriate part of the process as shown by the left-hand inside loop in *Figure 2.1*.

The flow chart in *Figure 2.1* also shows network and system considerations. In some instances, especially where significant delay reductions can be expected, additional traffic may be attracted. On the other hand, where some degree of congestion is involved, drivers and other users of the facility may be forced either to accept the situation or to consider other travel options, such as using a different route, starting the usual trip earlier, or using a different mode of transportation. Since some of these traveller responses may be desirable, the constrained intersection operations may become an integral part of the transportation demand management strategy for the corridor or area, as indicated in the outside left-hand loop in *Figure 2.1*.

Evaluation stage

The purpose of the signal evaluation stage is to assess whether and to what extent the objectives of the design or planning process have been met, what problems may be expected, and the quality of service provided to the users of the intersection. The Guide identifies a number of evaluation criteria and the procedures for determining them.

These evaluation criteria are broadly classified as *capacity related, queueing related* and *other criteria*. They include measures that can be directly perceived by drivers, cyclists and pedestrians, such as delay or the number of stops, as well as criteria needed to assess system parameters or impacts, such as capacity, queue reach, or pollutant emissions.

No recommended thresholds of the vast majority of the evaluation criteria are given in the Guide. It is the analyst's task to determine the planning, design or operational measures and values appropriate for a given set of conditions and identified problems.

2.5 Surveys

In order to provide the basis for comparison of existing operations with the expected benefits of the design, it is advisable to establish the current values of the evaluation parameters by surveys. *Chapter 6* provides a set of survey procedures for the most important input and evaluation parameters. They include arrival flow, saturation flow, overflow factor, average overall delay, average stopped delay and queue reach.

CHAPTER 3

ANALYSIS

This chapter describes the various parameters that are used to analyze the performance of a signalized intersection. These address traffic flow, saturation flow values, signal timing and phasing considerations, and other factors.

3.1 Traffic Flow

Traffic flow is defined as the number of vehicles, passenger car units, or pedestrians passing over a given cross-section of a roadway during a unit of time.

Although the unit of time usually taken is one hour, an identified flow rate may exist for shorter periods of time. For example, if 500 vehicles crossed the stop line of a lane in the first 30 minutes and 300 in the second, the *flow* during the first 30 minutes was 500 x (1.0 / 0.5) = 1000 veh/h. In the second 30 minutes, the flow was 300 x (1.0 / 0.5) = 600 veh/h. This is distinctly different from traffic *volume* that, in the above example, was 500 + 300 = 800 veh/h. Since steady traffic conditions may last for shorter or longer periods than a full hour, the Guide almost exclusively uses the term *flow,* with an hour as the time dimension. The difference between the two terms is especially important in the determination of total person delay.

The set of procedures in the Guide requires the determination of all flow parameters on a *lane-by-lane* basis. The allocation procedures are identified in this Section. Flows that may discharge during more than one phase must also be allocated to individual phases. These procedures are discussed in Section 4.2.1 on page 4-83.

The terms *arrival flow, departure flow,* or *person flow* should not be confused with *saturation flow, which is* the maximum rate of vehicle discharge from an accumulated queue after the beginning of the green interval. Saturation flow is defined in Section 3.2.1 on page 3-23.

3.1.1 Arrival and departure flows

Arrival flow is the number of vehicles or pedestrians per unit of time approaching the intersection during the design or evaluation period. Typically, the vehicular arrival flow is counted upstream of the end of an intersection approach queue (*Figure 3.1*). Only arrival flow should be used to represent travel demand at the intersection for the analysis, design, or evaluation.

The relationship between arrival flow and capacity is illustrated in *Table 3.1 on page 3-14*. Under traffic conditions well below saturation, the departure flow equals the arrival flow.

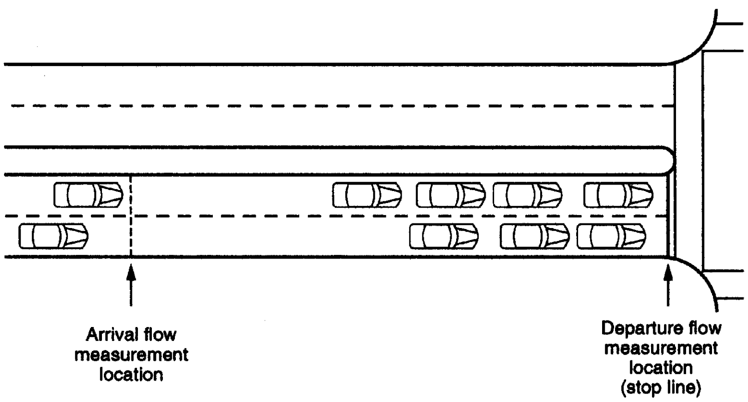


Figure 3.1 Vehicular arrival and departure flow

If dedicated lanes for each intersection movement exist, the arrival flow for left-turn, straight-through and right-turn movements can be directly surveyed. If such geometric conditions are not available, the arrival flow for individual intersection movements may be derived from the approach arrival flow, allocated in proportion to the departure flow from intersection counts for left-turn, straight-through and right-turn movements.

3.1.2 Units of vehicle flow

Vehicular traffic flows for all intersection movements and lanes can be expressed either as the number of vehicles per unit of time or as the number of passenger car units per unit of time. Most of the procedures in the Guide use passenger car units (pcu) to represent flow.

Table 3.1 Relationship among arrival flow, capacity and departure flow[1]

Arrival flow and capacity	Arrival flow and departure flow
$q_{arr} < C$	$q_{arr} = q_{dep}$
$q_{arr} \geq C$	$q_{dep} = C$ $q_{arr} - q_{dep}$ = rate of queue growth

1. Where: q_{arr} = arrival flow for a given lane upstream of the queue influence (units/time)
C = capacity of that lane (units/time)
q_{dep} = departing flow crossing the stop line (units/time).

Flow in vehicles per hour (veh/h)

This type of flow is expressed as a sum of all vehicles:

$$q = \Sigma_k q_k$$

where:

q = arrival or departure flow in a given lane (veh/h)

q_k = flow of vehicles of category k in a given lane (veh/h).

Flow in passenger car units per hour (pcu/h)

Vehicular traffic flow is more commonly expressed as a homogeneous entity by converting the individual vehicle categories into passenger car units (pcu).

Then:

$$q = \Sigma_k f_k q_k$$

where:

q = arrival or departure flow in a given lane (pcu/h)

q_k = flow of vehicles of category k in a given lane (veh/h)

f_k = passenger car unit equivalent of a vehicle category k (pcu/veh).

Passenger car units are normally approximated by an "average" passenger vehicle that can transport up to nine persons and has no more than four tires. Passenger cars, vans and pick-up trucks are usually included in the "car" category, although a separate value is also given.

Typical conversion factors for signalized intersections in Canada are shown in *Table 3.2 on page 3-15*.

Where specific counts by heavy vehicle types are not available, a combined passenger car unit equivalent of 2.0 may be used as an approximate value for trucks and buses.

Table 3.2 Passenger car unit equivalents[1]

Vehicle category	Passenger car unit equivalent (pcu/veh)
Passenger cars, vans, pick-up trucks	1.0
Single unit trucks	1.5
Multi-unit trucks	2.5
Multi-unit trucks heavily loaded	3.5
Buses	2.0
Articulated buses or streetcars	2.5
Motorcycles	0.5
Bicycles[2]	0.2 to 1.0
Pick-up trucks and vans[3]	0.9

1. **Sources:** Teply 1981, Hamilton 1986, Ottawa-Carleton 1994
2. Depending on the facility, bicycle flow and other traffic (Section 3.4.2 "Bicycles" on page 3-70)
3. If used as a category in mixed traffic

Peak Hour Factor

The peak hour factor is a measure of the variability of traffic flow over an hour. It is defined as follows:

$$PHF = \frac{\text{Hourly Volume}}{\text{Maximum rate of flow}}$$

Using a standard 15-minute analysis period:

$$PHF = \frac{\text{Hourly Volume}}{\text{Maximum 15-minute volume} * 4}$$

The maximum value for the PHF is 1.0, which represents a situation in which traffic flows are constant over the hour.

The PHF can be used to estimate the maximum hourly flow rate within an hour using:

$$q_{adj} = \frac{q}{PHF}$$

3.1.3 Person flow and vehicle occupancy

It is sometimes meaningful to represent transportation demand on the basis of person flow rather than vehicular flow. Examples include important public transit routes, high occupancy vehicle lanes, and locations that require transit priority measures. To convert vehicular flow to person flow it is necessary to consider the occupancy of each vehicle category:

$$q_{per} = \Sigma_k \, q_k O_k$$

where:

q_{per} = person flow rate (person/h)

q_k = traffic flow in vehicles in each category per hour (veh/h)

O_k = average occupancy of vehicles category k (person/veh).

Considerable fluctuation in vehicle occupancy may take place within a vehicular evaluation time or design period. Moreover, vehicle occupancies in different directions of travel may also vary. The applicable flow rate and the evaluation time relevant to person flows for individual vehicle categories, especially for public transit, should therefore be carefully considered. The time unit may therefore not always be one hour. In these instances, volumes of vehicles, instead of vehicle flows, must be used. (See “Traffic Flow” on page 3-13 and “Analysis period, evaluation time, design period, period of congestion, and transit assessment time” on page 3-17.) The resulting units are persons during the evaluation time or during the transit assessment time.

The process example 1 in Chapter 6 illustrates the procedure.

3.1.4 Pedestrian flow

In this Guide, pedestrians are considered separately from persons in or on vehicles.

Crosswalks and sidewalks adjacent to the intersection are the locations where pedestrian flows are considered in the analysis. In general, the procedures in this Guide deal with two-way pedestrian flows. Normally, no distinction between the directions in which the pedestrians are moving in the crosswalks is made. The flows are expressed as ped/h.

3.1.5 Analysis period, evaluation time, design period, period of congestion, and transit assessment time

Since most of the procedures in the Guide assume steady arrival flow conditions, one of the major decisions involves the *analysis period* during which such unchanged conditions can reasonably be considered. “Typical” conditions that are usually taken into account are, for example, the vehicular morning traffic peak, mid-day off-peak, evening peak, night-time off-peak, etc. Such steady conditions may last less than a full hour in small communities or longer than an hour in larger cities. The analysis period is also normally used as the *evaluation time* for which the measures of effectiveness are determined.

Some of the design and evaluation parameters are very sensitive to the time period during which the conditions prevail, especially in traffic conditions close to or over saturation. Local observations are therefore highly recommended. In the absence of such information, Table 3.3 on page 3-17 may be used. The *design period* need not necessarily be identical to the analysis period, evaluation period or the period of congestion. For instance, intersection approaches leading from industrial areas are often somewhat oversaturated during periods shorter than 15 minutes following the end of the afternoon shift. Although it may be appropriate to analyze and evaluate such conditions, using a 10-minute period as a basis for the design would normally not be considered logical.

Table 3.3 Suggested analysis periods and evaluation time

Regional population or other description	Analysis period or evaluation time t_e (min.)
< 100,000	15 to 30
100,000 to 500,000	30 to 60
> 500,000	60
special events	actual duration
shift change in industrial areas (regardless of regional population)	15 to 30

Where oversaturated conditions exist, the evaluation time is typically less than the *period of congestion.* While the evaluation period represents a steady state of traffic with a steady average arrival flow, the period of congestion involves vehicular traffic conditions with a build-up and dissipation of long queues. Such a situation implies that the mean of the arrival flow has changed during this period at least once. See “Queue at the end of evaluation period” on page 4-111.

Transit peaking characteristics are often different than traffic fluctuations. Person flow evaluation may therefore use a different time base, referred to as the *transit assessment time*. See “Non-vehicular delay” on page 4-105.

3.1.6 Flow allocation to lanes

Having more than one lane to choose from for the intended intersection movement, drivers tend to select the lane which has the shortest queue. If they had previous experience at that location, they may select the lane with the highest chance of not being delayed. Lane flow ratio can be employed as a surrogate for drivers’ decisions and is used as the parameter underlying the allocation of the arrival flows to individual lanes and movements. The following section outlines the considerations involved in the procedures.

Approaches without shared lanes

Flows are assigned to lanes by individual movements. If only one exclusive lane exists for the movement, all flows are assigned to this lane. If multiple exclusive lanes exist for one movement, flows should be assigned in proportion to the adjusted saturation flows for each lane. Additional lane flow imbalances, such as those caused by high pedestrian flows, should also be considered.

Approaches with shared lanes

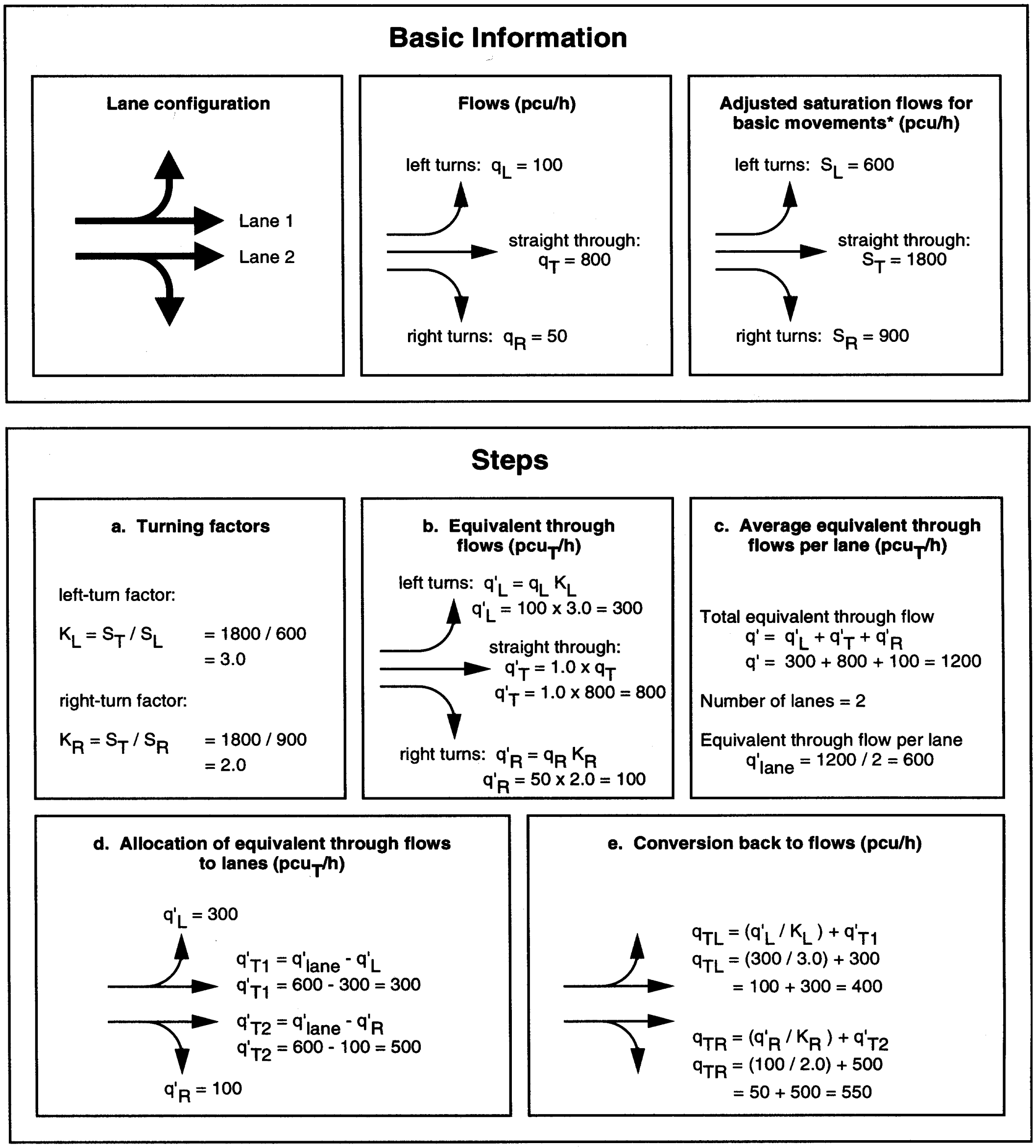
Counts or observations should be employed to allocate the individual movements to the available lanes. If this is not possible, the procedure outlined in the example in *Figure 3.2 on page 3-19* is appropriate.

Where one of the departure movements is assigned only to exclusive lanes, the allocation of flows for this movement proceeds as for approaches without shared lanes. (See “Approaches without shared lanes” on page 3-18.) The remaining directional flows are allocated to the shared lanes using the procedure in *Figure 3.2*.

Note that for the calculation, the flows are expressed in “equivalent through passenger car units” (pcu_T/h) that correspond to passenger car units for the straight through movement. All units of this equivalent through flow have identical headway requirements, but at the end of the procedure are converted back into the original passenger car units for each movement.

This calculation must yield logical results or values close to observed flows. The results may indicate that the lane is effectively functioning as an exclusive lane or that the initial assignment of the movements to the lanes is inadequate.

Other flow allocation procedures are possible (Akcelik 1989). The outlined method may then be used as a starting iteration. For instance, after the evaluation stage, equal delays or equal probabilities of discharge overload within individual movements may be applied as the flow allocation rule in a series of additional steps.



Note: *Saturation flows if lanes were operating as exclusive lanes

Figure 3.2 Example of flow allocation to an approach with shared lanes.

3.1.7 Special flow considerations

In addition to departure flows that occur during green intervals, there are also flows that take place during other portions of the cycle. They are the left and right turns during intergreen periods, right turns on red (RTOR) and, where permitted, left turns on red from one-way to one-way streets.

Left turns on intergreen period (LTOI)

Some drivers waiting in an exclusive lane for an opportunity to cross the opposing vehicular traffic stream during a permissive left-turn phase will not be able to make their turn until after the end of the green interval. The number of left turning passenger car units per cycle that may clear through an intersection during an intergreen period is a function of the intersection width or the size of the "storage" area, and local driver behaviour. With the exception of high traffic pressure situations, this number is largely independent of the duration of the amber interval or the all-red period. Examining local driver behaviour is advisable in these cases since higher values may influence the start lag of the following phase. The values observed in Canada are listed in the *Table 3.4 on page 3-20*. The crosswalk width is not included in the dimensions of the waiting space in the intersection.

The hourly left-turn flow on intergreen is then determined by multiplying the average number of passenger car units discharging during one cycle by the number of cycles per hour:

$$q_{LTOI} = n\ X_{LTOI}$$

where:

q_{LTOI} = left-turn flow on intergreen (pcu/h)

n = number of cycles per hour (cycle/h) = 3600 / c

X_{LTOI} = average number of left-turn passenger car units per intergreen period (pcu/cycle)

c = cycle time (s).

In shared left-and-through lanes with a permissive left-turn movement, the number of left turns on intergreen is usually conservatively assumed to be zero, unless a specific value is established by direct measurements.

Table 3.4 Average number of left-turning passenger car units that can discharge during one intergreen period

Number of cross-street lanes or metres available for the waiting vehicles	pcu / intergreen
1 lane or up to 5 m	0.5 to 1.0[1]
2 lanes or 5 m to 9 m	1.0 to 2.0
3 or more lanes or more than 9 m	2.0 to 3.0

1. The higher of these values is appropriate where the left-turn demand is high or where congestion exists.

Right turns on intergreen period (RTOI)

The number of right-turning passenger car units per cycle that may clear through an intersection during the intergreen period is a function of intersection geometry. For example, exclusive right-turn lanes with or without a right-turn island have different operating characteristics. It is also a function of conflicting movements during the intergreen period, such as the number of pedestrians or opposing left-turn traffic still moving through the shared areas. Unlike left turns, the "storage" area at the near side of the intersection does not have a major influence on the number of right-turn passenger car units that can discharge during the intergreen period.

Depending on local driver behaviour and pedestrian flows in the adjacent crosswalk, the number of passenger car units discharging during one intergreen period of an ending phase can be taken on average as 1.0 to 2.0 (Teply 1990). The hourly flow is then determined by multiplying the average number of passenger car units, discharging during one cycle, by the number of cycles per hour:

$$q_{RTOI} = n\, X_{RTOI}$$

where:

q_{RTOI} = right-turn flow on intergreen (pcu/h)

n = number of cycles per hour (cycle/h) = 3600/c

X_{RTOI} = average number of right-turn passenger car units per intergreen period (pcu/cycle)

c = cycle time(s)

Right turns on red interval (RTOR)

Most Canadian jurisdictions allow right turns during the red interval. The exception until recently has been the Province of Quebec. However, Quebec now permits RTOR in most locations, except on the island of Montreal. This type of movement is similar to right turns at stop-sign controlled intersections. The utilization of right turns on red is primarily a function of the conflicting vehicular and/or pedestrian flows, and the presence of a right-turn lane for the approach on the left (*Figure 3.3*). The turning radius, the width of the lanes, and sight triangles may also have some effect.

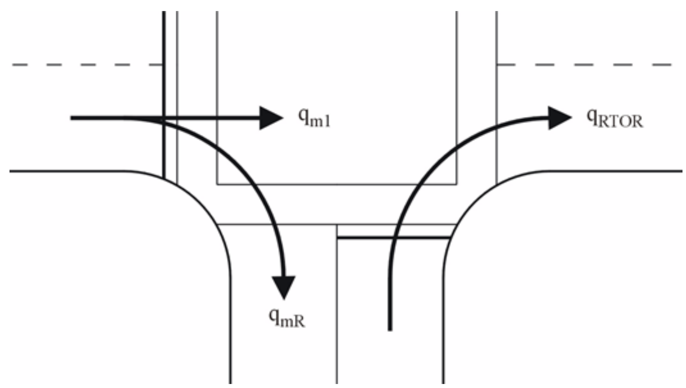


Figure 3.3 Basic flow and geometric conditions for right-turn-on-red flow (RTOR) estimation.

The number of right turns on red interval is very limited in shared right-and-through lanes and is therefore usually not considered unless the right-turn flow is very high. In that case, the shared right-and-through lane may operate as an exclusive right-turn lane.

Figure 3.4 illustrates the linear relationship that may be used to approximate the hourly right-turning flows. Canadian research indicates an upper flow limit of 700 to 900 pcu/h for instances where drivers making the right turn do not encounter traffic on the main road (Poss 1989, Stewart and Hodgson 1994). These flows are comparable to values measured at stop signs. The calculation takes into account the rate of the conflicting flow in the right lane plus, to a lesser degree, the impact of the right-turning vehicles in the conflicting phase. The process involves four steps as shown in *Table 3.5* .

Table 3.5 Determination of right-turn-on-red flows[1]

Step	Equation
1	$q'_{m1} = q_{m1}\ c\ /\ r$
2	$q'_{mR} = q_{mR}\ c\ /\ r$
3	$q'_{m} = q'_{mR}\ /\ 2 + q'_{m1}$
4	$q_{RTOR} = 850 - 0.35\ q'_{m}$

1. where:

q'_{m1} = through flow *rate* in the curb lane of the conflicting approach during the red interval for the subject approach (pcu/h) (Figure 3.1 on page 3-14)

q_{m1} = through flow in the curb lane of the conflicting approach during the red interval for the subject approach (pcu/h) (Figure 3.1 on page 3-14)

c = cycle time (s)

r = red interval when right turns on red from the subject approach can take place (s)

q'_{mR} = right-turn flow *rate* from the curb lane of the conflicting approach during the red interval for the subject approach (pcu/h)

q'_{m} = effective priority flow *rate* of the conflicting approach during the red interval for the subject approach (pcu/h)

q_{mR} = right-turn flow in the curb lane of the conflicting phase during the red interval for the subject approach (pcu/h). Where the curb lane of the conflicting approach is fully dedicated to the right-turn movement or where it is separated from the main part of the intersection by an island, $q_{mR} = 0$

q_{RTOR} = maximum right-turn flow on red interval from the subject approach (pcu/h). Right turns on intergreen period (page 3-20) are not included in this value.

Higher flow values may be expected in situations where the subject right-turn lane is separated with an island and controlled by a yield sign but do not have an exclusive discharge lane. A true free flow channelized right turn is not controlled by the signal and therefore should not be analyzed using the methods outlined in the Guide. On the other hand, a high pedestrian flow across the subject approach may substantially reduce the q_{RTOR} value.

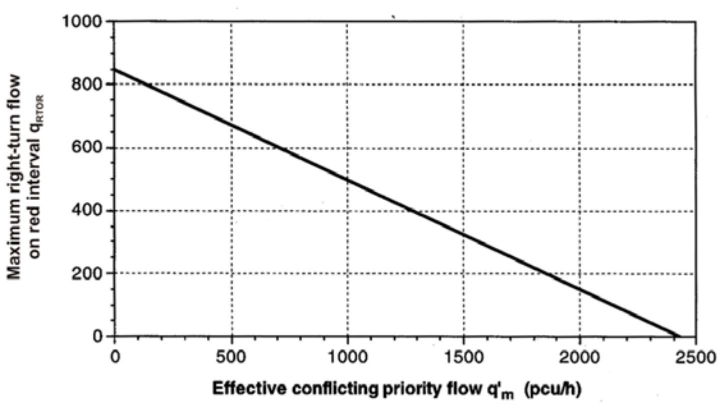


Figure 3.4 Flow rates during right turns on red as a function of the conflicting traffic direction.[1]

1. **Source:** Stewart and Hodgson 1994

Left turns on red interval (LTOR)

Most jurisdictions allow left turns from a one-way roadway to another one-way roadway during red intervals. The calculation process is similar to the determination of right turns on red interval.

3.2 Saturation Flow

While the vehicular *arrival flow* represents the travel demand, *saturation flow* is the underlying variable that determines stop line capacity. This is the capability to accommodate the vehicular arrival flow, measured by individual intersection lanes. It is the highest sustainable departure flow across the stop line during the green interval. Saturation flow forms the basis for the calculation of many parameters that describe how well the intersection operates.

Since the Guide employs almost exclusively a set of lane-by-lane procedures for design and performance evaluation, saturation flow must be determined separately for each lane. This method makes it possible to consider special conditions for each lane, and allows a fair comparison of the arrival flow with the capacity to accommodate it.

3.2.1 The concept of saturation flow

Saturation flow is a fundamental macroscopic vehicular traffic characteristic, which reflects the impact of interrupted vehicular flow inherent in signal operations. It is defined as the rate at which vehicles that have been waiting in a queue during the red interval cross the stop line of a signalized intersection approach lane during the green interval. Saturation flow is usually expressed in passenger car units per hour of green (pcu/h). A vehicle is considered "discharged" when its front axle passes the stop line. These reference points are consistent with the usual definition of *headway* on free-flow facilities.

Saturation flow can be measured directly in the field, or estimated from a basic regional value using the adjustments for specific local conditions outlined in the following Section of the Guide. These have been updated from the 2nd Edition. Caution is advised when using or comparing saturation flows from the literature since the measurement techniques and reference points on the road and on the vehicle vary. As a result, the quoted values may not be compatible with the procedures discussed in the Guide.

An approximate conversion formula between the saturation flows used in the Guide and in the Highway Capacity Manual is included in *"Relationship between Saturation Flow in this Guide and in the Highway Capacity Manual (HCM)"* on page 3-26 of this guide.

The basic saturation rates quoted in the Guide were derived from studies conducted across Canada. These have been updated for the 3rd Edition. Since saturation flow is the cornerstone of the analysis and design process, it is recommended that specific basic values for a given community or region be determined from field measurements. The recommended saturation flow survey method is described in Chapter 6.

Departure flows from a long queue at the stop line are shown as the histogram in *Figure 3.5*. In such fully saturated conditions, after the initial hesitation following the display of the green signal, vehicular traffic discharges at a nearly constant rate until shortly after the beginning of amber when a sharp drop occurs. The saturation flow concept is traditionally represented by a rectangular transformation of the saturated departure flow, as illustrated in *Figure 3.5*. This Figure is based on a classical source on the subject (RRL 1963). The Highway Capacity Manual (TRB 2000) and the Australian guidelines (Akcelik 1981) each use a different interpretation of the events. Many Canadian surveys indicate that the departure rate of traffic is not quite constant for very long green intervals. The saturation flow peak value usually drops after about 50 seconds of green (Teply 1981). This observation is consistent with international experience (OECD 1983) and reflected the procedures described in "Duration of green interval" on page 3-39.

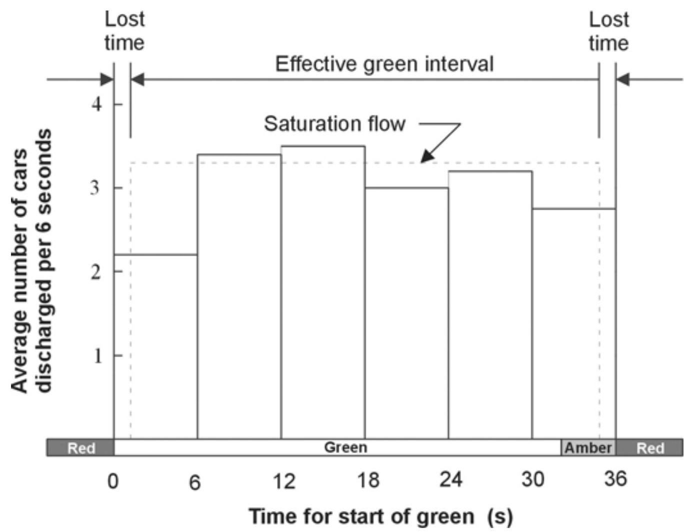


Figure 3.5 Saturation flow concept as applied in the Guide. Adapted from RRL 1963

Figure 3.6 shows an example of a measured saturation flow in two different formats. The histogram corresponds to the concept illustrated in Figure 3.5 on page 3-24. The height of the bars represents the saturation flow value for each time increment. The line graph shows the saturation flow in the "cumulative average" format for a given portion of the green interval. For example, the value shown at 20 seconds after the start of green gives the average saturation flow for the first 20 seconds of the displayed green interval, the value at 30 seconds shows the average saturation flow for the first 30 seconds of the green interval, and so on. If the saturation remained constant for long green intervals, the cumulative representation would eventually asymptotically reach the same value as the average value of the histogram. The values from the cumulative graph for green intervals between 25 to 50 seconds may serve as a good approximation of the saturation flow.

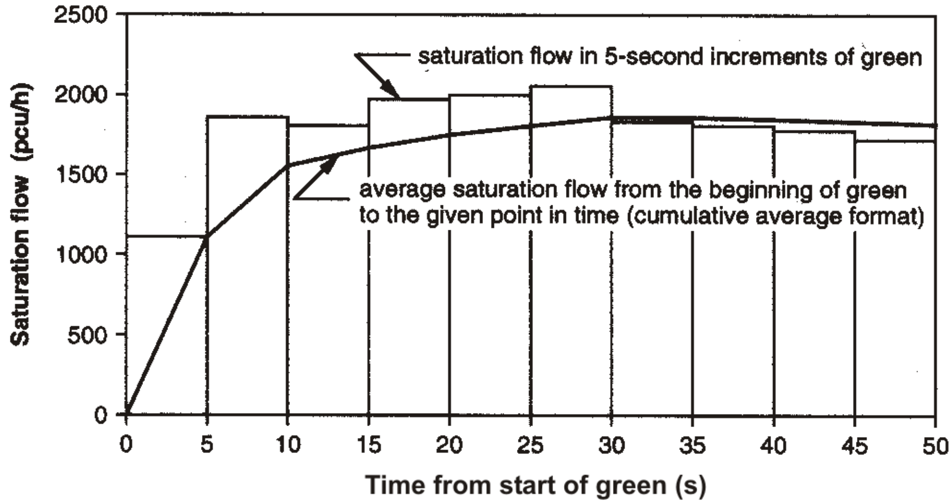


Figure 3.6 Typical example of measured saturation flow. (University of Alberta 1993.)

The cumulative average representation has been found to be effective for the analysis of surveyed data, especially for intersections with low degrees of saturation. Chapter 6 includes an example of the application of the procedure and illustrates the advantages of the cumulative average format.

3.2.2 Units of saturation flow

Similar to arrival flow, saturation flow is expressed as the number of passenger car units per unit of time or vehicles per unit of time, usually one hour. A qualifier per hour “of green” is often added since only the departures during green intervals are considered.

Saturation flow in pcu/h

Typical passenger car unit equivalents established at many signalized intersections in Canadian cities are listed in “Flow in passenger car units per hour (pcu/h)” on page 3-15. The resulting units of saturation flow are passenger car units per hour (pcu/h).

Saturation flow in veh/h

For several procedures in the Guide, flows and saturation flows must be input in vehicles per hour (veh/h) of green. Saturation flow in veh/h for the prevailing traffic composition can be determined from the regional value of the saturation flow in pcu/h using the equation below. The conversion assumes the same traffic composition as observed in the arrival flows for individual lanes:

$$S_{veh/h} = S_{pcu/h} / \Sigma (\%q_k f_k / 100)$$

where:

$\%q_k$ = % vehicles of category k in the vehicular arrival flow

f_k = passenger car unit equivalent for vehicles category k.

Relationship between Saturation Flow in this Guide and in the Highway Capacity Manual (HCM)

The 2000 Highway Capacity Manual defines the time reference for saturation flow surveys as the *front axle* passing over the stop line. Moreover, the measurement starts with the time of entry of the *fourth vehicle* in the queue. The definitions of the effective green interval in Section *3.3.2* and lost time in Section *3.3.4* in this Guide and in the HCM also differ. As a result, the saturation flow and the effective green interval determined by the HCM method is higher than its corresponding value measured by the procedure described in Chapter 5 (Teply and Jones 1991). The computational techniques in each document are consistent in the application of the respective definition, but the measured values are therefore not directly transferable between both documents. An approximate regression relationship between both saturation flow types in typical conditions has been determined during the work on the Guide as:

$$S_{HCM} = 1.05\ S_{CCG}$$

where:

S_{HCM} = approximate value of the saturation flow corresponding to the HCM method (veh/h). The vehicles in this case are passenger cars only.

S_{CCG} = saturation flow measured or calculated using the methods described in this Guide (pcu/h).

3.2.3 Basic saturation flow

The basic saturation flow reflects the departure rate of straight-through vehicular traffic flow at the stop line of a signalized intersection approach lane during the green interval under ideal geometric, pavement surface, traffic, and weather conditions in a given community. Since local conditions are rarely ideal, this saturation flow value cannot usually be applied directly. It is, however, useful for comparisons among regions and cities.

Saturation flow for less than ideal geometric conditions can be derived by the application of various adjustment factors described in the Guide. Weather conditions, pavement conditions and intersection environment must be considered separately.

It is practical to determine a set of saturation flow values for typical conditions in the community. They may be applied as a starting point for any analytical, planning, design or evaluation work.

Weather conditions

Many Canadian regions feature a climate in which “winter” driving conditions occur during a considerable portion of the year. As a result, they warrant separate consideration of winter saturation flows in the analysis and design process. Typical winter saturation flows are about 5% to 20% lower than summer saturation flows for identical geometric, pavement surface and traffic conditions (Teply 1977, Teply 1981). A similar reduction occurs during non-winter conditions when heavy rainfall makes road conditions difficult, and the spray behind moving vehicles obscures the driver's vision.

No reduction has been observed at signalized intersections in Canada when temperatures are above -10^{o}C and the road surface is dry.

Winter driving conditions in many regions feature temperatures below -10^{o}C, dry pavement or hard packed and well sanded snow. The effect of these conditions is often negligible in the south-central and maritime regions of Canada, but in the dry air of the prairie and northerly regions, the exhaust fumes and vapour obstruct close range visibility behind each vehicle. In all Canadian regions, temperatures above freezing with heavy rainfall or very wet roads with puddles cause substantial water spray and have a similar effect. Under these circumstances, the saturation flow values may be about 10% lower.

Extreme winter conditions are characterized by any of the following: heavy snowfall; blizzard; freezing rain at any air temperature; and a slippery pavement surface. In the prairie and northerly regions, even with clear sky and dry pavement, air temperature below -30^{o}C causes very significant reductions of visibility by exhaust fumes and vapour (Teply 1977). There are of course many areas of Canada and other countries at similar latitudes with a range of frequent weather conditions that can fall under the heading of extreme winter conditions. These conditions may cause extremely long headways and therefore low saturation flow values. They occur only infrequently in many larger urban centres and they are usually not considered in design or planning but may be explicitly included in the evaluation of specific circumstances for these areas.

In regions with long periods of *extreme winter* or *wet summer* driving conditions, it is advisable to determine the applicable saturation flow values. This value may not need to be used directly in the design, and it is useful for the assessment of difficulties that may be expected during less frequent, but typical, local climatic conditions.

Pavement conditions

Using adjustment factors in the Guide makes it possible to include the influence of various geometric elements of the intersection. The quality of the pavement surface, however, must be considered with the decision on the value of the saturation flow applied as a starting point for the adjustments. “Poor” pavement conditions may be characterized by a severely cracked surface, potholes, or deep ruts. It also includes temporary situations where the final layer of asphalt has not been laid and manhole covers or catch basin grates are raised above the surrounding pavement surface. Poor pavement conditions also apply to situations with streetcar or railway tracks in, next to, or crossing a travelled lane, regardless of the smoothness of ride in traversing the tracks.

Surveys in the Toronto and Edmonton regions indicate that such poor pavement conditions reduce the saturation flow that would otherwise correspond to other prevailing conditions by 10% to 15%. Under deteriorated pavement surface conditions with high density of potholes, this reduction may be substantially more severe. Similar conditions also occur during a large portion of the year in northern areas with snow-packed roads.

Quantification of pavement conditions is usually accomplished by measuring indicators such as roughness, plus the extent and severity of various types of pavement distress. Combined pavement condition scores have been calculated for applications in pavement management, and may be applied for developing corresponding saturation flow values for typical local pavement situations (RTAC 1977, RTAC 1987).

Intersection environment

Saturation flow at an intersection is influenced by the environment of individual approaches. For example, an approach with narrow sidewalks accommodating numerous pedestrians and with high-rise buildings exhibits a lower saturation flow than an approach with identical geometric parameters in a low land use density area with service roads and wide set-backs of light industrial buildings. It is useful to determine a set of standard saturation flow values for such typical situations in a region or an urban area.

These environments may include a range from high activity areas specific to central business district intersection approaches to low activity areas characterized by outlying industrial or suburban arterial roadways. Intersection environment classification applies to individual intersection approaches, not necessarily the whole intersection.

The criteria for the selection of these activity levels should include:

- Activities affecting vehicular traffic flow related activities, such as the presence of frequent bus stops, commercial deliveries, driveway turns, taxis or other drop-offs.
- Intensity of pedestrian activities and flows.

The values for straight-through lanes at typical arterial intersection approaches in newer suburban or industrial areas of most Canadian cities usually represent the basic saturation flow for the community or region for non-winter conditions.

Although direct measurements are preferable, approximate basic saturation flow values can be established on the basis of values derived in similar communities.

Table 3.6, Table 3.7, and *Figure 3.8* show typical saturation flow values measured in nine Canadian cities. These include values for exclusive through lanes and exclusive left turn lanes in both suburban and downtown locations. The second edition included data for exclusive through lanes only, for seven cities. The current edition includes data for most of those cities as well as additional locations. The data were collected between 2003 and 2005. In the case of Victoria, the Victoria Section returned to the same locations counted in the 1990s to produce a time series of data.

Based on this data, the average of the downtown values is equal to 93% of the average of the suburban values.

Table 3.6 Typical saturation flows for Canadian cities (pcu/h)

A: Through Movement

Approach environment	Victoria BC	Edmonton AB	Calgary AB	Regina SK	Windsor ON	Waterloo ON	Ottawa ON	Toronto ON	Montreal QC	Fredericton NB
Low Activity (suburban)	1735	1850	2100	1800	1720	1950	1827	1810	1870	1665
High Activity (downtown)	1565	1650			1685	1775	1749	1605	1785	

B: Left Turn Movement[1]

Approach environment	Victoria BC	Edmonton AB	Calgary AB	Windsor ON	Waterloo ON	Ottawa ON	Toronto ON	Montreal QC
Low Activity (suburban)	1631	1850	1875	1525	1725	1642	1850 (FAG)[2] 1740 (LTGA)[3]	1975
High Activity (downtown)	1565	1650		1685	1775	1749	1600 (FAG) 1270 (LTGA)	

1. The above values represent typical measured saturation flows, not necessarily those used by the administrations of individual cities in specific analytical, design or planning applications.
2. FAG: Flashing Advanced Green
3. LTGA: Left Turn Green Arrow

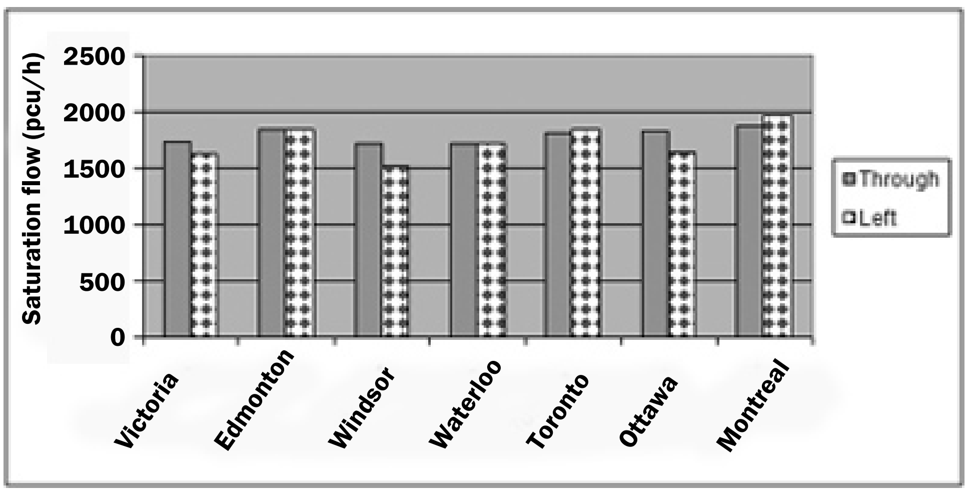


Figure 3.7 Typical values of saturation flows in Canadian cities (pcu/h)

Table 3.7 on page 3-30 following compares the values from the 2nd Edition to those collected for this edition. No single pattern is discernible in terms of the differences. This reinforces the need to collect and utilize current, locally-specific data whenever possible.

Table 3.7 Comparison of Saturation Flow Values, 2nd Edition to 3rd Edition

Approach Environment	Victoria	Vancouver	Calgary	Edmonton	Hamilton	City of Toronto	Ottawa
2nd Edition							
Low Activity (suburban)	1800	1850	1850	1750	1830	1870 to 1950	1815
High Activity (downtown)	1700		1750	1550	1650	1680 to 1750	1600
3rd Edition							
Low Activity (suburban)	1735		2100	1850		1850	1827
High Activity (downtown)	1565			1650		1650	1749
Difference (%)							
Low Activity (suburban)	-4%		14%	6%		-3%	1%
High Activity (downtown)	-8%			6%		-4%	9%

Figure 3.8 shows the saturation flow values in the cumulative format measured in five Canadian cities. For lanes with low degrees of saturation in Canadian communities, saturation flow may be approximated by inter- and extrapolation between these lines, starting from the values for short time increments (5 s or 10 s). See Figure 5.4 “Estimation of the saturation flow for situations where only a limited number of the green interval increments can be surveyed.” on page 5-132.

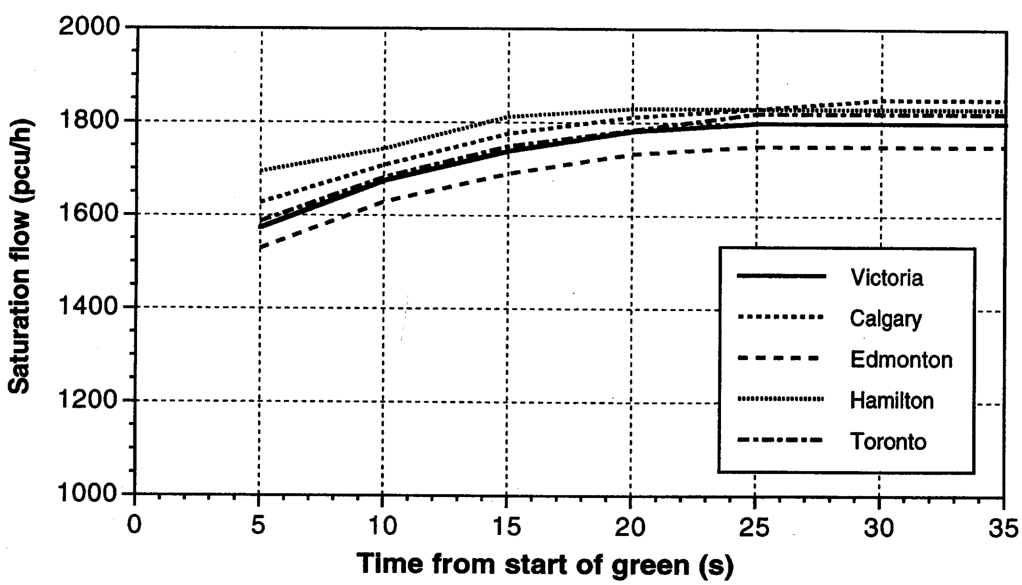


Figure 3.8 Typical measured saturation flows in Canadian cities in the cumulative average format.

3.2.4 Saturation flow adjustment factors

In the absence of directly measured saturation flows at the analyzed intersection, a number of adjustment factors must be considered and, if applicable, used to adjust the basic saturation flow values on each approach lane of the intersection. The adjustment factors included in this Guide reflect the influence of the following specific local conditions:

a. Geometric conditions

a. lane width
b. grade
c. turning radius
d. queueing and discharge space

b. Traffic conditions

a. transit stops
b. parking
c. pedestrians

c. Control conditions

a. duration of green interval
b. protected left turns
c. permissive left turns
d. permissive left turns with pedestrians
e. right turns with pedestrians
f. various shared lane combinations

The adjustment factors depend not only on the combination of intersection geometric, traffic, and control conditions, but depend also on the assignment of movements to individual phases and lanes (see "Phase composition and cycle structure" on page 3-55,) and allocations of flows to individual lanes (see 3.1.6 "Flow allocation to lanes" on page 3-18).

Accumulation of adjustment factors

The adjusted saturation flow depends on the basic saturation flow and is a function of the applicable adjustment factors:

$$S_{adj} = S_{basic} f(F_{adj})$$

where:

S_{adj} = adjusted saturation flow (pcu/h)

S_{basic} = basic saturation flow (pcu/h)

$f(F_{adj})$ = adjustment functions (Table 3.8 on page 3-32)

F_{adj} = individual adjustment factors.

As shown in Table 3.8 on page 3-32, in many instances the adjustment function is simply a multiplication of individual adjustment factors. In several cases, a combined factor that reflects the effect of the combination of several specific local conditions is provided. In some instances, the user must consider which of the factors plays a dominant role, and then decide whether to apply only one factor or some combination of all of them. The Guide provides advice for some of the most critical cases.

Table 3.8 Adjustment Factors for Saturation Flows

		A. Geometry				B. Traffic			C. Control					
		a. lane width	b. grade	c. turning radius	d. space	a. transit stops	b. parking	c. pedestrians	a. duration of green	b. protected LT	c. permissive LT	d. permissive LT + peds	e. RT + peds	f. shared lanes
A. Geometry	a. lane width		×	×	×	×	⊗	○	×	×	⊗	⊗	○	×
	b. grade	×		×	×	×	×	○	×	×	⊗	⊗	○	⊗
	c. turning radius	×	×		○	⊗	×	○	×	○	○	○	●	○
	d. space	×	×	○		○	⊗	○	⊗	○	○	○	○	○
B. Traffic	a. transit stops	×	×	⊗	○		○	○	⊗	○	○	○	○	○
	b. parking	⊗	×	×	⊗	○		○	○	○	○	○	○	○
	c. pedestrians	○	○	○	○	○	○		⊗	○	○	●	●	○
C. Control	a. duration of green	×	×	×	⊗	⊗	○	⊗		●	○	○	○	○
	b. protected LT	×	×	○	○	○	○	○	●		•	•	•	○
	c. permissive LT	⊗	⊗	○	○	○	○	○	○	•		•	•	○
	d. permissive LT + peds	⊗	⊗	○	○	○	○	●	○	•	•		•	○
	e. RT + peds	○	○	●	○	○	○	●	○	•	•	•		○
	f. shared lanes	×	⊗	○	○	○	○	○	○	○	○	○	○	

• Not applicable

○ Judgement required - examine dominant effects

× Multiplication of factors

⊗ Possibly a multiplication of factors

● Special procedure or recommendation - refer to corresponding Section

3.2.5 Adjustments for geometric conditions

Lane width

Table 3.9 and *Figure 3.9* illustrate the adjustment functions and show that for lane widths in the range of 3.0 to about 4.4 m no adjustment is required. If the approach width varies, but not sufficiently to add extra lanes, the width at the narrowest point within 30 m of the stop line should be used.

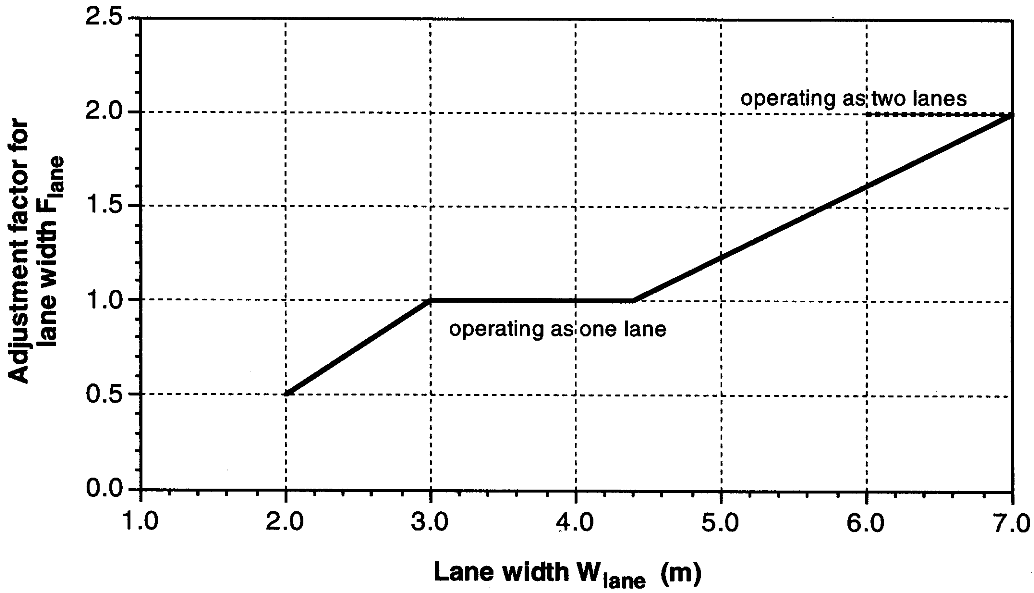


Figure 3.9 Saturation flow adjustment factor for lane width.

Exit lanes should be at least as wide as the approach lanes. If this condition is not met, the width of the exit lane should be used. Through vehicles will avoid using an approach lane that does not continue on the exit side of the intersection. If the length of a lane for through vehicles is limited either on the approach or exit side, the limited queueing or discharge space adjustment may apply (see "Queueing and discharge space" on page 3-36). The adjustment factor is determined from the equations in *Table 3.9*.

Table 3.9 Lane width adjustment factor

Lane width W_{lane} (m)	Adjustment factor F_{lane}
$W_{lane} \le 3.0$ m[1]	0.5 W_{lane} - 0.5
3.0 m < $W_{lane} \le 4.4$ m	1.0
4.4 m < $W_{lane} \le 6.0$ m	0.385 W_{lane} - 0.695
6.0 m < $W_{lane} \le 7.0$ if this part of the approach functions as **one lane**	0.385 W_{lane} - 0.695
6.0 m < $W_{lane} \le 7.0$ if this part of the approach functions as **two lanes**[2]	2.0

1. Lanes narrower than 2.75 m should be used only in exceptional cases.
2. In most instances, lanes wider than 6.0 m will operate as two lanes. Local investigation is advisable.

Grade

It is not necessary to apply the gradient factor unless an obvious or visible grade exists on the approach (usually more than +2% or less than -2%). The effect of grade is more significant during winter or wet conditions.

The adjustment factor for lanes with grade is calculated as follows:

$$F_{grade} = 1 - (G + HV)$$

where:

F_{grade} = saturation flow adjustment factor for the effect of grade

G = average approach grade within 50 m upstream of the intersection (% /100)

HV = proportion of heavy vehicles, such as buses, trucks or recreational vehicles in the vehicular arrival flow. The arrival flow is *not* converted to passenger car units (% /100).

For *downhill* approaches, the effect of heavy vehicles is usually negligible (with the exception of very steep grades). Therefore, HV = 0. Since the sign of the grade is negative, the resulting factor for downhill approaches is greater than 1.0, but should be constrained to a maximum of 1.1.

Some judgment is required where all of the *uphill* slope on the approach occurs within a short distance upstream of the stop line and the green interval is short relative to the time required to discharge the vehicles queued on the grade (Figure 3.10 on page 3-34). The effect of the grade is more pronounced compared to situations where the green interval is long relative to the time required to clear the vehicles queued on the grade. An uphill grade on discharge lanes may also affect the operation, especially in cases where heavily loaded trucks are present.

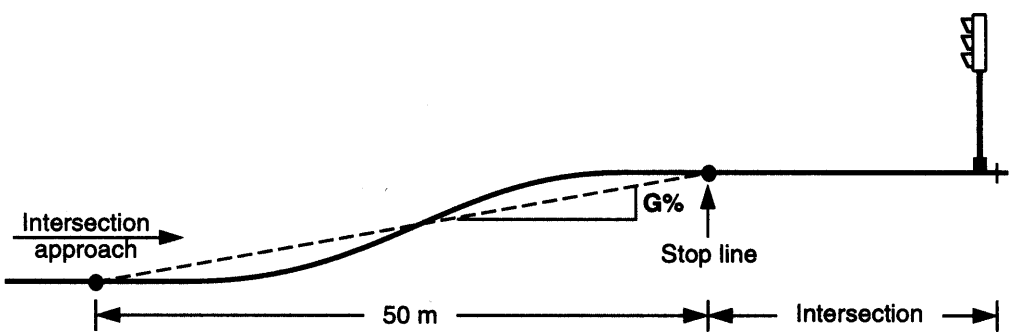


Figure 3.10 Example of an intersection approach with a short uphill grade.

Turning radius

The turning radius adjustment is usually applied to right-turn lanes and shared right-turn-and-through lanes only, although it may also affect left turns from a one-way street to another one-way street. It does not usually apply to permissive or protected left turns that are governed by different aspects of driver behaviour (see sections from “Protected left turns in exclusive lane” on page 3-40 to “Other left-turn situations” on page 3-45), with the exception of left turns at intersections of two-way two-lane roads.

Table 3.10 and *Figure 3.11* illustrate the calculation of the adjustment factor for radius.

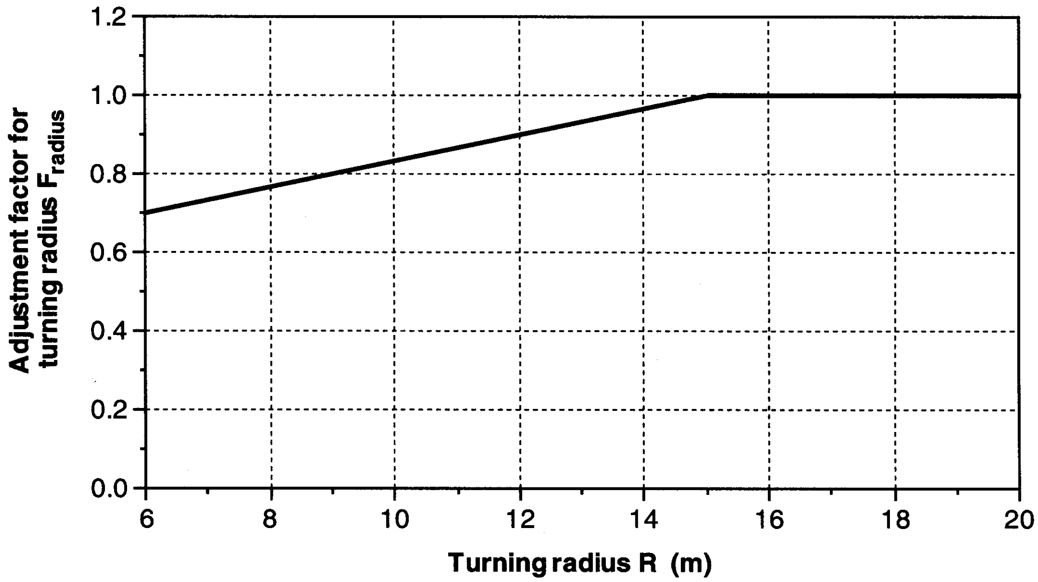


Figure 3.11 Saturation flow adjustment factor for turning radius.

The combination of vehicular right turns with pedestrians is described in Section 3.2.11 "Right turns in exclusive lanes" on page 3-46. The effects of pedestrians may override the radius effect (Teply 1990). Long trucks turning into a narrow roadway have a significant influence on saturation flow.

Table 3.10 Calculation of the adjustment factor for turning radius

Turning radius R (m)	Adjustment factor F_{radius}
R < 15.0	0.5 + R / 30
R ≥ 15.0	1.0

Queueing and discharge space

If inadequate storage or discharge space prevents a queue of vehicles from clearing an intersection without impedance, regardless of whether the blockage is caused by geometric conditions or parked vehicles on the approach or in the discharge space (*Figure 3.12*), adjustments must be made to the saturation flow values of the affected lanes. The calculation process for reduced queueing or discharge space may affect the curb lane and the second lane differently.

This section describes the technique for the determination of the saturation flow for exclusive lanes. The determination of saturation flows for shared lanes is described from "Shared right-turn and through lane" on page 3-49 to "Special lanes" on page 3-52.

The adjusted average saturation flows for the curb and the second lanes (*Figure 3.12*) can be calculated in a series of steps shown in Table 3.11 on page 3-36:

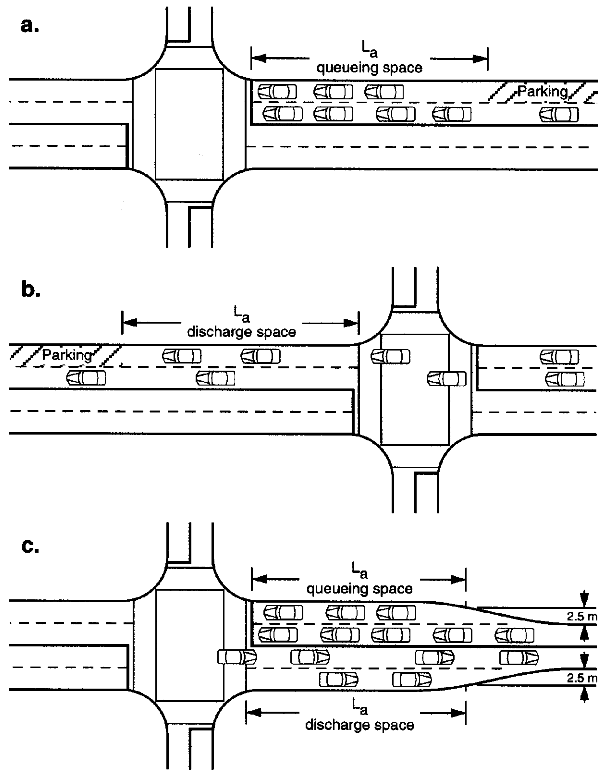


Figure 3.12 Examples of limited queueing or discharge space configurations.

Table 3.11 Calculation of the saturation flow adjustment factor for limited queueing or discharge[1]

Step	Calculation	Note
1	$L_r = S\, g_e\, L_{pcu} / 3600$	
2	if $L_r \le L_a$	this adjustment is not necessary
3	if $L_r > L_a$ calculate the ratio as: $u_L = L_a / L_r$	
4	$F_{1queue} = [u_L + r\,(1 - u_L)]$	adjustment factor for the curb lane
5	$F_{2queue} = [u_L + (1 - r)(1 - u_L)]$	adjustment factor for the second lane

1. Where: L_r = storage required to accommodate all passenger car units during the green interval (m)
S = saturation flow adjusted for other factors, such as lane width, grade, etc. (pcu/h)
g_e = effective green interval (s)
L_{pcu} = average length required for a passenger car unit in a queue, usually taken as 6.0 m
L_a = available storage (queueing) or discharge distance (*Figure 3.12*) (m)
r = proportion of vehicles using the curb lane. After the departure of vehicles queued within the available storage length, the arrival flow for both lanes is supplied by one approach lane only. The value of r depends on lane continuity, and will range from 0 to1.0. As an example, when one approach lane supplies flows equally to two lanes, r = 0.5.

3.2.6 Adjustments for traffic conditions

Transit stops

This saturation flow adjustment is similar to the adjustment for limited storage or discharge space, except that these spaces are restricted only while a bus or a streetcar is present at the stop. Normally, the impact of transit stops on saturation flow is significant only when transit vehicles regularly block traffic lanes during the green interval or its portion (Jacques and Yagar 1994). Where the total expected dwell times are less than 5% of the total green time in an hour, a transit stop adjustment is normally not considered.

Some system considerations that may influence the operation are mentioned in Sections 3.4.3 "Transit vehicles" on page 3-71 and 4.5.1 "Signal coordination and other system considerations" on page 4-93. Reserved bus lanes are discussed in "Special lanes" on page 3-52.

Near-side transit stops

This transit adjustment factor applies to bus and streetcar stops and is calculated as:

$$F_{transit} = 1.0 - k\ B\ T\ /3600$$

where:

$F_{transit}$ = adjustment factor for near-side bus or streetcar stops

k = coefficient for the effect of transit vehicle loading during the green intervals, calculated as:

$k = [(\%\ \text{loading on green})\ {*c}]\ /\ [100{*}\ g_e]$

Where all loading during the evaluation time occurs during the red intervals, the effect of the bus or streetcar stop is negligible

$k = 0$;

where all loading takes place during the green intervals

$k = c\ /\ g_e$.

B = number of transit vehicle arrivals per hour (bus/h, streetcar/h)

T = average transit dwell time during the evaluation period determined as the average boarding and alighting time for a transit vehicle including deceleration and acceleration time (s). In the absence of direct measurement, it can be estimated using an average boarding and alighting time of 1.5 to 2 s for one passenger plus about 6.0 s for the deceleration and acceleration.

c = cycle time (s)

g_e = effective green interval (s).

Streetcars operating in mixed traffic in the centre lanes of a street require special consideration. Where no on-street streetcar loading platform exists adjacent to the streetcar lanes, streetcars influence the operation of the lane with the rails and the lane(s) between the streetcar and the curb. Streetcars have been implemented in more cities in recent years, with a range of operating strategies from mixed traffic to exclusive lanes. The Cities of Toronto, Ottawa, Calgary and Edmonton are among the Canadian cities which have existing streetcar or light rail transit systems, and they may have additional information to share regarding the effect of streetcars on traffic operations.

Far-side bus stops

The situation shown in *Figure 3.13* applies to bus stops only. The saturation flow adjustment factor F_{bus} that approximates the effect of a far-side bus stop is calculated as shown in *Table 3.12*.

Table 3.12 Calculation of the adjustment factor for far-side bus stops[1]

Step	Calculation	Note
1	$t_d = T\ B$	
2	$t_{rs} = (L_a / L_{pcu})\ (3600 / S)\ B$	
3	if $t_d - t_{rs} \leq 0$	this adjustment is not applicable
4	if $t_d - t_{rs} > 0$	
5	$F_{bus} = 1 - [(t_d - t_{rs}) / 3600]$	

1. Where: t_d = total bus dwell time in an hour (s/h)
T = average dwell time of a transit vehicle at a stop determined as the average boarding and alighting time for the transit vehicle plus the acceleration and deceleration time (s)
B = frequency of service (bus/h)
t_{rs} = time to fill the curb lane behind a bus at the stop determined as the time recovered due to the available storage (s/h)
L_a = length of the available storage space (m), *Figure 3.13*
L_{pcu} = length of the space occupied by one passenger car unit, usually taken as 6 m
S = saturation flow adjusted by other factors, such as width or grade (pcu/h)
F_{bus} = saturation flow adjustment factor for a far-side bus stop (pcu/h)

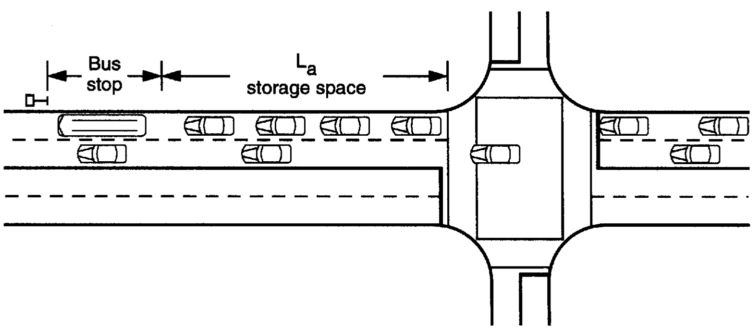


Figure 3.13 Example of the far-side bus stop saturation flow configuration.

Parking interference

In addition to the obvious geometric effects of on-street parking, such as the reduced effective approach width or limitation of the queueing and discharge space, there is a “frictional” component introduced by vehicles moving into and leaving the parking stalls, car door openings and the generally cautious behaviour of drivers and cyclists as they proceed along the lane adjacent to parked vehicles.

The adjustment factor for the lane closest to the parked vehicles is given by:

$$F_p = 0.90 - 0.005\ N_m$$

where:

F_p = adjustment factor for parking

N_m = number of parking manoeuvres per hour within 50 m upstream or downstream of the stop line.

During those times when on-street parking is not permitted and no illegal parking is tolerated:

$F_p = 1.00$

"Shared lane with limited queueing or discharge space" on page 3-51 deals with two-lane approaches with left turns where parking impedes the queueing or storage space.

Pedestrian traffic

Because the saturation flows in Table 3.6 on page 3-28 are based on average conditions with typical pedestrian traffic flows for a given type of environment, no special adjustment is usually necessary for the effect of pedestrians on through lane saturation flows. Corrections for the influence of pedestrian traffic on left-turn saturation flow are detailed in "Permissive left turns in exclusive lane with pedestrians" on page 3-44, and on right-turn saturation flow in "Right turns in exclusive lanes" on page 3-46.

3.2.7 Adjustments for control conditions

These conditions, represented by the structure of the cycle, composition of individual phases and individual green intervals, have a major impact on saturation flow. Their influence is usually combined with the effect of geometric and traffic conditions.

Duration of green interval

The first five to seven vehicles crossing the stop line during the initial portion of a green interval usually require longer headways than during the later parts of the green interval. As a result, the maximum saturation flow has not yet fully developed. Moreover, where long green intervals are fully saturated, some drivers in the long queues become less attentive and do not start moving immediately after the preceding vehicle. Their headways are therefore also longer than the minimum saturation flow headway during the steady flow portion of the green interval and, consequently, the saturation flow declines.

Canadian saturation flow surveys indicate the necessity to adjust the basic saturation flow in relation to the duration of the green interval. The need is emphasized by the approximation of the effective green interval and lost times, and facilitated by the representation of the measured values in the cumulative average format. The adjustment factor is illustrated in Figure 3.14 on page 3-40 and its values may be determined from *Table 3.13* below.

Green intervals for protected left-turn phases may not require any adjustments other than those described in "Protected left turns in exclusive lane" on page 3-40.

Table 3.13 Calculation of the adjustment factor for the duration of the green interval

Duration of green interval g (s)	Saturation flow adjustment factor for green interval F_{green} [1]
$g \le 20$	$0.833 + g / 120$
$20 < g \le 50$	1.0
$50 < g < 60$	$1.5 - g / 100$
$g \ge 60$	0.9

1. Note: the displayed, not the effective, green interval is applied.

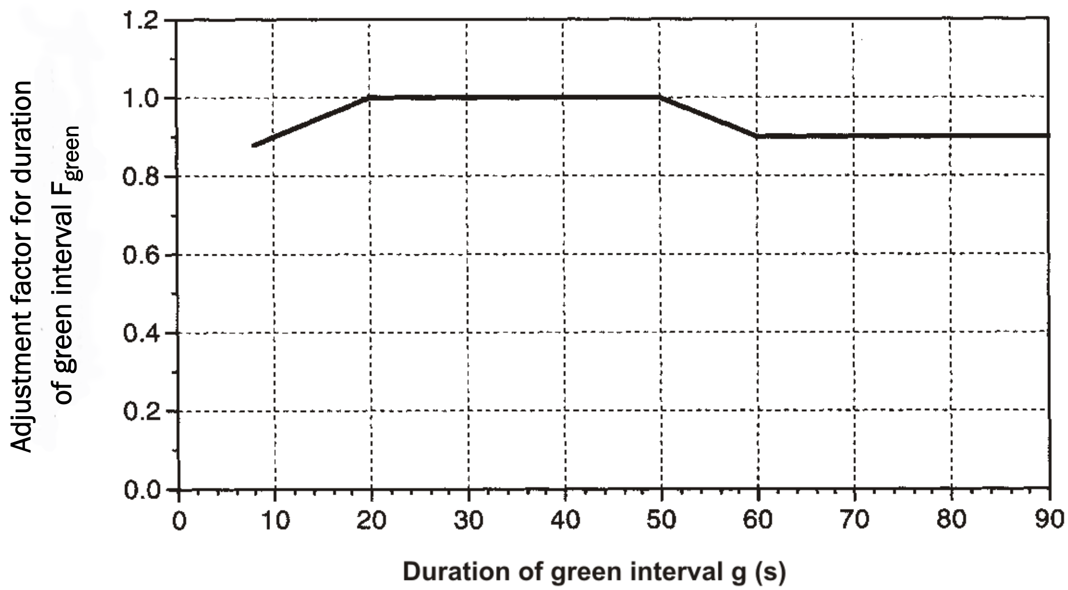


Figure 3.14 Saturation flow adjustment factor for green interval.

Protected left turns in exclusive lane

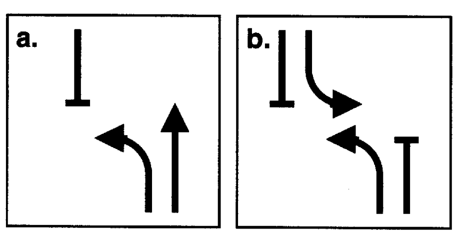


Figure 3.15 Typical left-turn protected phases.

In these cases the signal phase allows left turns while all vehicular traffic in the opposing direction is stopped or only the opposing left turns are allowed (*Figure 3.15*). The pedestrian crosswalk that must be traversed by left-turning vehicles displays a Don't Walk signal indication. This Section assumes that a dedicated left-turn lane with sufficient queueing space is available. "Protected left-turn and through movements" on page 3-44 describes adjustments for shared lanes and "Double exclusive left turn lanes" on page 3-45 discusses those turns.

The effect of turning radius is usually negligible. Because drivers can see several turning vehicles ahead and have some "escape" space available, they tend to follow closer. The saturation flow may therefore be higher than for straight through lanes, especially for short green intervals operating at capacity. *Figure 3.16 A* and *Figure 3.16 B* as well as *Table 3.14* show a comparison of the saturation flow values for straight-through and protected left-turn movements in dedicated lanes measured in several Canadian cities. Data for low pedestrian activity and high pedestrian activity areas is presented in the figure. The level of pedestrian activity does not appear to have a clearly definable impact on left turn saturation flow relative to through values. The level of pedestrian activity does have an effect overall, however, which can be seen comparing Figures 3.16 A and B.

Protected left-turn saturation flow for *under-saturated* conditions is usually more variable than the saturation flow for straight through movements. Where no measurements are available, the left-turn saturation flow may be taken as the basic saturation flow adjusted for lane width, grade and the duration of the green interval, if applicable.

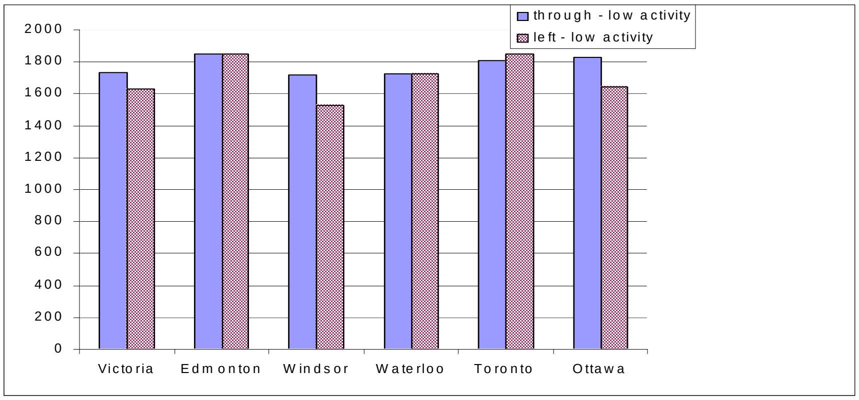


Figure 3.16 A Comparison of saturation flows for straight-through and protected left-turn movements in dedicated lanes for suburban areas (low pedestrian activity).

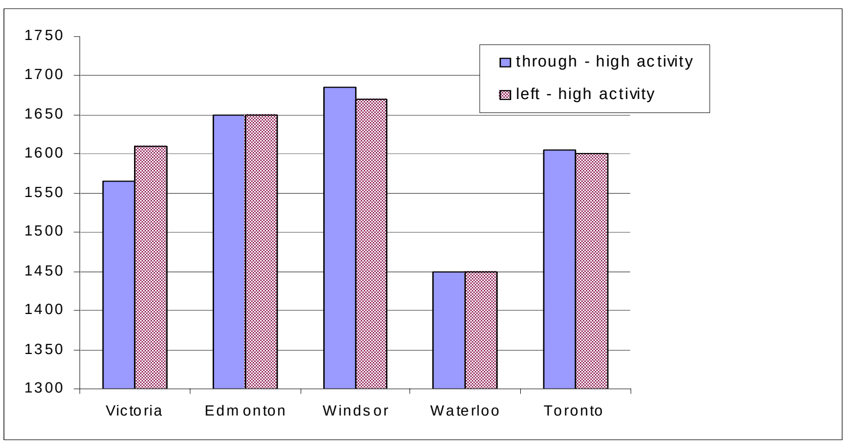


Figure 3.16 B Comparison of saturation flows for straight-through and protected left-turn movements in dedicated lanes for suburban areas (high pedestrian activity).

The protected left turn saturation flow in *saturated* conditions, where a continuous left-turn queue exists, may be greater than the basic saturation flow for straight-through movements. The protected left-turn saturation flow adjustment factor for these instances is:

$$F_L = 1.05$$

No adjustment for the duration of the green interval is necessary. Very short green intervals may feature saturation flows higher by more than 5% of the basic saturation flow as evidenced by the Toronto and Ottawa surveys (*Figure 3.16 A*, *Table 3.14*).

Table 3.14 Saturation flow values for protected left-turn movements under saturated conditions in dedicated lanes (pcu/h)[1]

Year	Approach environment	Victoria	Vancouver	Calgary	Edmonton	Windsor	Waterloo	Ottawa	City of Toronto
Prior to 1995 (2nd Edition)[2]	Low activity (suburban)	1650	1650	1650	1650 to 1750	n/a	n/a	1650 to 1900	1850 to 2000
2004 - 2005	Low activity (suburban)	1631	n/a	1875	1850	1525	1725	1642	1850 (FAG) 1740 (LTGA)
2004 - 2005	High activity (downtown)	1609	n/a	n/a	1650	1670	1450	n/a	1600 (FAG) 1500 (LTGA)

1. Only dedicated lanes with protected left-turn movements under non-winter conditions and with good pavement conditions are included, regardless of the signal display practice (such as flashing green or solid / flashing green arrows).
2. Data from 1993 to 1995

Protected left turn movements are typically operated as leading phases (i.e. starts before the beginning of the through green phase for the opposing direction of travel), however, there are circumstances when a lagging phase (i.e. starts after the end of the through green phase for the opposing direction of travel) may be acceptable. The saturation flow characteristics of the lagging protected phase are analogous to a leading protected phase

3.2.8 Permissive left turns in exclusive lane

Permissive left turns in exclusive lane without pedestrian flow

Saturation flow for left turning vehicular flow that penetrates through vehicular traffic in the opposing direction depends mostly on the availability of sufficient gaps in the opposing vehicular traffic flow and the willingness of drivers to accept gaps to complete the left-turn manoeuvre.

The calculation proceeds in two steps as shown in *Table 3.15*. The resulting adjustment factor (*Figure 3.17*) is applied to the basic saturation flow adjusted for lane width and grade. The effect of the turning radius is usually negligible. Situations where pedestrian flow is present in the crosswalk are discussed in “Permissive left turns in exclusive lane with pedestrians” on page 3-44.

Table 3.15 Determination of the saturation flow adjustment factor for permissive left turns[1]

Step	Calculation	Note
1	$q'_o = q_o\, c / g_e$	average effective rate of opposing flow during the green interval
2	$F_L = 1.05\, e^{(-0.00121\, f\, q'_o)} - 0.05$	

1. Where:
   q'_o = rate of opposing flow during the green interval (pcu/h)
   q_o = opposing flow (pcu/h)
   c = cycle time (s)
   g_e = effective green interval for the opposing traffic flow (s) during the period when left turns are permitted. If the opposing flow received any benefits of leading or delayed protected phasing, its corresponding portion must be excluded. All other opposing flows in all lanes considered for the f-coefficient below are included.
   f = coefficient that reflects the effect of the number of opposing flow lanes (*Table 3.16*). It includes flows in:
   - all through lanes
   - shared through-and-right-turn lanes.

   It does not include flows in exclusive left-turn lanes, the left-turn flow in a shared left-turn and straight-through lane, flow in exclusive right-turn lanes with a right-turn island.
   F_L = left-turn adjustment factor.

Table 3.16 Effect of the number of lanes on permissive left-turn saturation flow rate[1]

Number of opposing lanes	1	2	3	4
f	1.0	0.625	0.51	0.44

1. **Source:** Richardson 1982

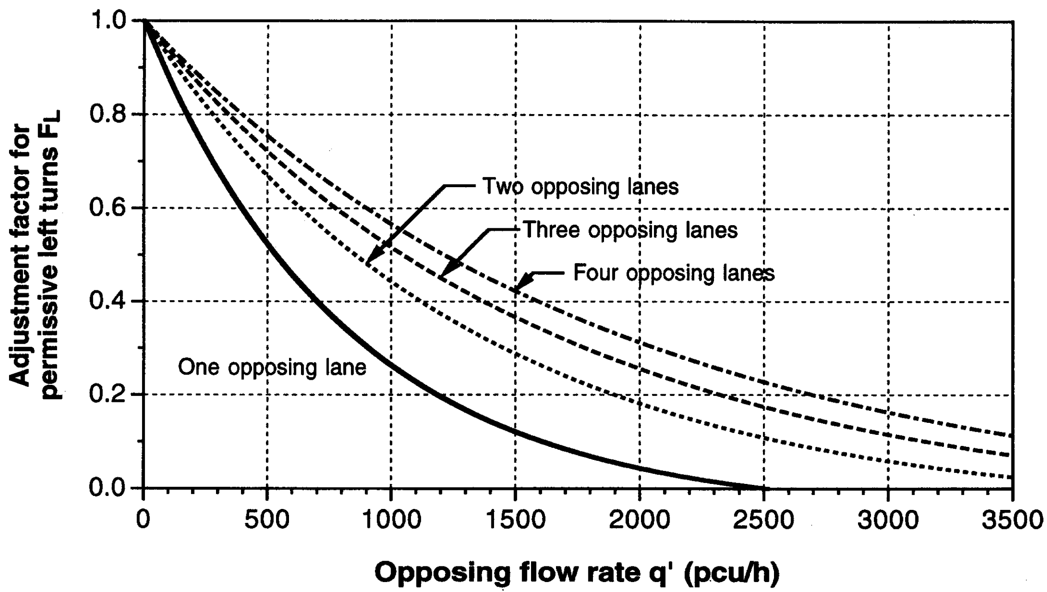


Figure 3.17 Left-turn saturation flow adjustment factor as a function of the opposing traffic flow rate during green and the number of lanes.

Permissive left turns in exclusive lane with pedestrians

The effect of pedestrian flow using the left-hand crosswalk is usually small. The rate of pedestrian flow is usually highest during the walk interval and the initial portion of the pedestrian clearance interval. This coincides with that portion of the green interval when left-turning vehicles must wait for the initial queue of opposing vehicular traffic flow to discharge. If the pedestrian flow rate were included, it might result in significant underestimation of the vehicular left-turn saturation flow.

In situations where the whole interval that is available for left turns is used by a continuous heavy pedestrian flow, the saturation flow is usually very low, frequently negligible. Its estimate can be obtained by adding the *rate* of this pedestrian flow to the *rate* of the vehicular flow, after it has been multiplied by the coefficient for the effect of the number of lanes. The procedure in "Permissive left turns in exclusive lane without pedestrian flow" on page 3-42 is then followed. The units of the combined "opposing flow rate" are a somewhat unusual combination of passenger car and pedestrian "units", but the resulting saturation flow values are realistic in most instances.

3.2.9 Shared left-turn and through lane

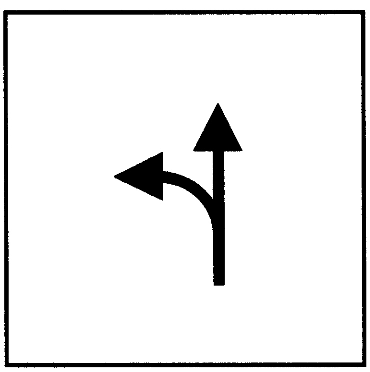


Figure 3.18 A shared left-turn and through lane.

Protected left-turn and through movements

With this operation (*Figure 3.18*), saturation flows can be taken as those for dedicated through lanes from the saturation flow values shown in Table 3.6 on page 3-28,unless local observations indicate otherwise.

Permissive left-turn and through movements

The saturation flow is determined using the proportions of flows expressed in passenger car units and equivalent through passenger car units. The resulting saturation flow value therefore applies only to the specific flow allocation situation. The procedure is described in *Table 3.17*. Prior to the calculation, arrival flows must be allocated to individual lanes ("Approaches with shared lanes" on page 3-18.)

Table 3.17 Determination of saturation flow adjustment for lanes with permissive left-turn and through movements[1]

Step	Calculation	Note
1	Determine the saturation flow for the left-turn movement as if the lane were an exclusive left-turn lane	3.2.8 "Permissive left turns in exclusive lane" on page 3-42
2	Calculate the left-turn movement factor $K_L = S_T / S_L$	"Approaches with shared lanes" on page 3-18 Figure 3.2 "Example of flow allocation to an approach with shared lanes." on page 3-19 Step a
3	Determine the equivalent through lane flow for the left-turn movement $q'_T = K_L q_L + q_T$	"Approaches with shared lanes" on page 3-18 Figure 3.2 "Example of flow allocation to an approach with shared lanes." on page 3-19
4	$F_{TL} = (q_L + q_T) / q'_T$	

1. Where:
K_L = left-turn movement factor (*Figure 3.2*)
F_{TL} = saturation flow adjustment factor for the shared left-turn and through lane
q_L = left-turn flow in the shared lane (pcu/h)
q_T = straight-through flow in the shared lane (pcu/h)
q'_T = equivalent through flow for the shared lane (pcu_T/h)
The F_{TL} factor is applied to the lane saturation flow for the straight through movement
S_T = saturation flow for straight through flow (pcu/h)
S_L = saturation flow for left turn flow (pcu/h)

3.2.10 Other left-turn situations

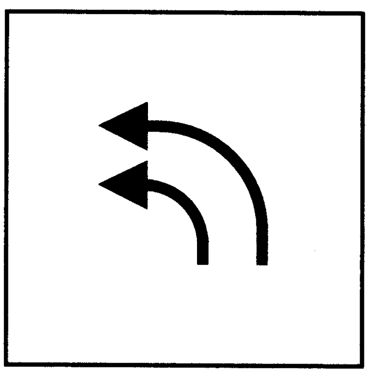


Figure 3.19 Lane designations for double left-turn lane arrangement. First left-turn lane = the extreme left lane adjacent to the median or centre line.

Double exclusive left turn lanes

Left-turn saturation flows for double left-turn situations (*Figure 3.19*) should be determined by the procedures outlined in "Protected left turns in exclusive lane" on page 3-40 for protected phasing. For the jurisdictions that allow the operation of dual left turn lanes with permissive phasing, see "Permissive left turns in exclusive lane with pedestrians" on page 3-44. Local driver experience may indicate otherwise. In situations where only a few such signalized lanes exist in the region, it is likely that the second left-turn lane will have a reduced saturation flow, potentially up to 20% less than the first lane.

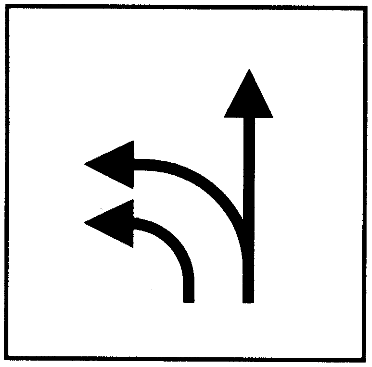


Figure 3.20 Lane arrangement for an exclusive left-turn lane with a shared left-turn and through lane.

Permissive movements from an exclusive left-turn lane with a shared left-turn and through lane

Such double left turn situations have an exclusive left-turn lane plus a shared left-turn and through lane (*Figure 3.20*). The movements take place during a permissive left-turn phase in which the left-turning drivers must penetrate the opposing flow. The procedures outlined in "Permissive left turns in exclusive lane" on page 3-42 and "Permissive left-turn and through movements" on page 3-44 apply. For a sequence of protected / permissive or permissive / protected left-turn phases, the saturation flows must be determined separately for each phase using the procedures outlined in "Protected left turns in exclusive lane" on page 3-40 for protected and "Permissive left turns in exclusive lane" on page 3-42 for permissive phasing. Since these procedures require arrival flow allocation to lanes and phases, an iterative process starting with estimated values may be required.

Left turns with restrictions downstream

Significant reductions of saturation flow have been observed at entrances to parking garages, at 'tight diamond' interchanges, at underpasses, tunnel entrances, or other situations where left-turning drivers enter a physical or psychological bottleneck.

If the drivers in the left-turn lane have their view of the approaching opposing traffic obstructed by the vehicles of the opposing left-turning movement, or their downstream sightlines are obstructed, the left-turn saturation flow may be as much as 30% lower.

Where the lanes of the discharge intersection leg are narrow or where the turning radius is less than 12 m with significant truck and bus movements, or where the turning angle is greater than 90^o, saturation flows may be as much as 30% lower than their basic value.

3.2.11 Right turns in exclusive lanes

Right turn flows on intergreen and right turns on red are described in "Right turns on intergreen period (RTOI)" on page 3-20 and "Right turns on red interval (RTOR)" on page 3-21. Where the right turn lane is controlled by traffic signals, adjustments for the radius may apply. The lane usually carries the full basic saturation flow, similar to a through lane, during the green or green arrow signal indication, unless pedestrians are present on the unsignalized portion of the crosswalk. Where such a lane is separated from the intersection by a channelized island and an exclusive discharge lane, no adjustments are necessary, and the right turns can be eliminated from the signal analysis.

Right turns with Pedestrians

In many urbanized areas right-turn vehicular flows are severely affected by pedestrians using the parallel crosswalk during the same phase, as shown in *Figure 3.21*. Empirical functions developed in three Canadian cities are illustrated in Table 3.18 on page 3-47 and Figure 3.22 on page 3-48. They can be used to determine the right-turn saturation flow penetrating the adjacent pedestrian flow.

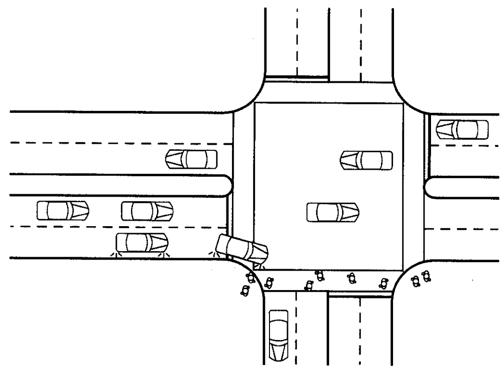


Figure 3.21 Right turns with pedestrians

Table 3.18 Calculation of saturation flow adjustment factor for right turns crossing pedestrian flow[1]

Step	Note	Calculation
1	calculate the average pedestrian flow rate during the phase	$q'_{ped} = q_{ped}\ c / g$
2	if $q'_{ped} \leq 200$ and the intersection is in an area with few pedestrians on sidewalks	$F_{Rped} = 1.0$
3	calculate the adjustment factor using one of these functions (*Figure 3.22*) (or a locally developed function)	Toronto: $F_{Rped} = 0.60 - q'_{ped} / 8516$ Edmonton: $F_{Rped} = 0.44 - q'_{ped} / 9320$ Vancouver: $F_{Rped} = 0.44 - q'_{ped} / 14100$

1. Where:
   q'_{ped} = approximate average pedestrian flow *rate* during the walk interval and pedestrian clearance period (ped/h)
   q_{ped} = two-way pedestrian flow on the crosswalk (ped/h)
   c = cycle time (s)
   g = green interval (s). Note that the displayed green interval is applied.
   F_{Rped} = saturation flow adjustment factor for right turns with pedestrians.

In areas of higher pedestrian activity (i.e. pedestrian flow rate higher than 200 ped/h), some pedestrians walking on sidewalks may suddenly enter the crosswalk, and drivers therefore proceed cautiously. The adjustment factor is calculated from the appropriate function in step 3 of Table 3.18 on page 3-47 with the pedestrian flow rate put in as zero. On the other hand, where few pedestrians are present or sidewalks are not provided, pedestrians have virtually no effect on the right-turn saturation flow. The pedestrian flow rate threshold of 200 ped/h may be adjusted to specific local conditions.

The applicability of the Toronto, Edmonton or Vancouver function depends on local driver behaviour. For instance, Edmonton features lower saturation flows than Toronto for the same pedestrian flow rates and, for higher pedestrian flow rates, lower than Vancouver. The size of the municipality, vehicular traffic "pressure" and the degree of respect for pedestrians may guide the judgment. In Halifax, for example, drivers typically stop for pedestrians wherever they cross. Saturation flow data from Halifax is unfortunately not available at present. If it becomes available, it should be representative of trends as North American Cities become more pedestrian supportive. Right turns on intergreen period (page 3-20) are not included.

Tight radii have no effect on right-turning vehicular saturation flows over crosswalks where pedestrians are present or are expected by the drivers to be present. The adjustment factor for the radius (Section 3.2.5 on page 3-33) therefore does not normally apply in conjunction with the adjustment factor for right turns with pedestrians in downtowns or similar environments.

In industrial and other areas, the adjustment factors for pedestrians and radius may be multiplied, provided the conditions of Step 2 of *Table 3.18* are met.

Some municipalities delay the beginning of the walk interval by several seconds after the start of the vehicular green interval at intersections with high pedestrian and high right-turn flows. This portion of the green interval is excluded and should be treated as a right-turn flow without pedestrians.

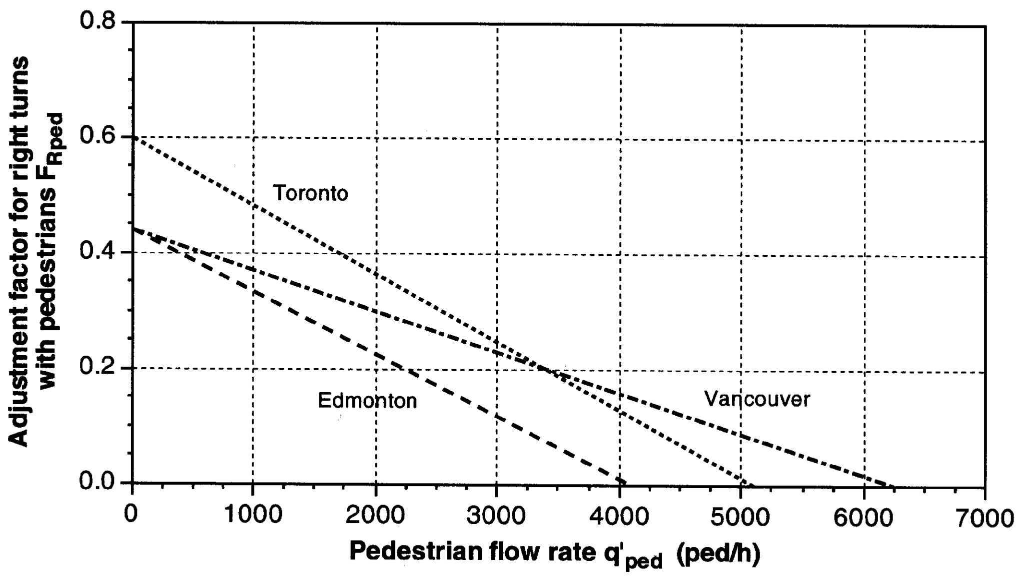


Figure 3.22 Saturation flow adjustment for right turns crossing pedestrian flow in three cities. **Sources:** Richardson 1982, Poss 1985, Teply 1990, Vancouver 1993

Double right turns

Unless regional driver behaviour indicates otherwise, the initial saturation flow value for each lane of a double right turn is considered to be the same. This configuration is rarely used.

Right turns with restrictions downstream

Where right-turning vehicles are entering a roadway with tight geometric features, restricted clearance, or other physical or psychological bottlenecks, the saturation flow for right-turn lanes or shared right-turn and through lanes may be as much as 30% lower. The reduction is more severe where a significant number of long vehicles turns right.

3.2.12 Other lane situations

Shared right-turn and through lane

The saturation flow for this situation (*Figure 3.23*) is determined using the proportions of flows expressed in passenger car units and *equivalent through* passenger car units, similar to shared left-turn and through lanes. The resulting saturation flow value therefore applies only to the specific flow situation. The procedure is described in *Table 3.19*.

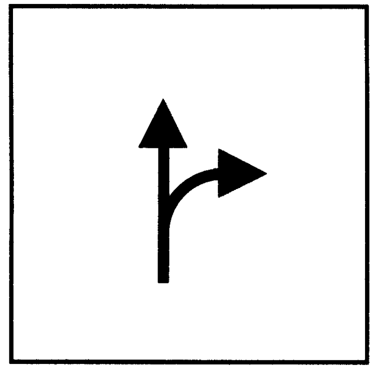


Figure 3.23 Shared right-turn and through lane.

Table 3.19 Calculation of the saturation flow adjustment factor for shared right-turn and through lanes[1]

Step	Calculation	Note
1	Determine the saturation flow for the right-turn movement as if the lane were an exclusive right-turn lane	Section *3.2.11*
2	Calculate the right-turn movement factor $K_R = S_T / S_R$	Section *3.1.6* step a. *Figure 3.2*
3	Determine the equivalent through lane flow for the shared lane $q'_T = K_R\, q_R + q_T$	Section *3.1.6* *Figure 3.2*
4	$F_{TR} = (q_R + q_T) / q'_T$	

1. Where:
K_R = right-turn movement factor Section *3.1.6*
q_R = right-turn flow in the shared lane (pcu/h)
q_T = straight-through flow in the shared lane (pcu/h)
q'_T = equivalent through flow for the shared lane (pcu_T/h)
F_{TR} = saturation flow adjustment factor for the shared right-turn and through lane (pcu/h)
The resulting saturation flow adjustment factor is applied to the saturation flow for the straight-through movement
S_T =saturation flow for straight through flow (pcu/h)
S_R = saturation flow for right turn flow (pcu/h)

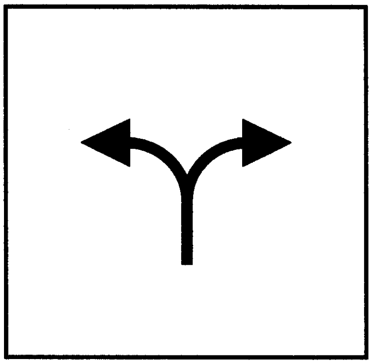


Figure 3.24 Shared left-turn and right-turn lane at T-intersections.

Shared left-turn and right-turn lane at T-intersections

Similar to other shared lane combinations, the saturation flow for this lane arrangement shown in *Figure 3.24* is determined using the proportions of *flows* expressed in passenger car units and equivalent passenger car units that require the same time to discharge. The straight-through flow is not present in this lane but its basic saturation flow value may still be used for the determination of the equivalent through passenger car units. The resulting saturation flow value applies only to the specific split between the two flows. The procedure follows the steps similar to those described in “Shared left-turn and through lane” on page 3-44 and “Shared right-turn and through lane” on page 3-49 and are illustrated in Table 3.20 “Calculation of the saturation flow adjustment factor for a shared left-turn and right-turn lane” on page 3-50.

Where pedestrians are present on the crosswalks, the adjustment factor for right turns with pedestrians may be used as a reasonable approximation.

Table 3.20 Calculation of the saturation flow adjustment factor for a shared left-turn and right-turn lane[1]

Step	Calculation	Note
1	determine the saturation flow for the left-turn movement as if the lane were an exclusive left-turn lane (S_L)	Section *3.2.8*
2	determine the saturation flow for the right-turn movement as if the lane were an exclusive right-turn lane (S_R)	Section *3.2.11*
3	calculate the right-turn movement factor $K_R = S_T / S_R$	Section *3.1.6* *Figure 3.2*
4	calculate the left-turn movement factor $K_L = S_T / S_L$	Section *3.1.6* *Figure 3.2*
5	determine the equivalent through flow $q'_T = q_R K_R + q_L K_L$	Section *3.1.6* *Figure 3.2*
6	$F_{LR} = (q_R + q_L) / q'_T$	

1. Where: K_R = right-turn movement factor (Section *3.1.6*)
K_L = left-turn movement factor (Section *3.1.6*)
q_R = right-turn flow in the shared lane (pcu/h)
q_L = left-turn flow in the shared lane (pcu/h)
q'_T = equivalent through flow for the shared lane (pcu_T/h)
F_{LR} = saturation flow adjustment factor for the shared left-turn and right-turn lane.
S_T = saturation flow for straight flow (pcu/h)
S_L = saturation flow for left turn flow (pcu/h)
S_R = saturation flow for right turn flow (pcu/h)

The resulting saturation flow adjustment factor is applied to the saturation flow that would exist in the subject lane for the straight through movement, possibly adjusted for lane width and grade.

Saturation flow values for these situations may range from the basic saturation flow to as low as 1200 pcu/h, depending on local geometric conditions and on regional driver behaviour. Special investigations are advisable.

Combined left-turn / through / right-turn lane without opposing vehicular flow and without pedestrians

This lane movement combination (*Figure 3.25*) is used at one-lane approaches with a special phase or, exceptionally, on narrow one-way streets. The directional volume split may have a significant impact. It is advisable to simplify this situation by calculating the saturation flow adjustment as if this lane were a left-turn and through lane, or a right-turn and through lane, depending on the heavier turning volumes.

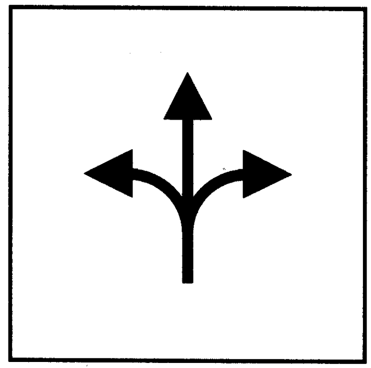


Figure 3.25 Combined left-turn / through / right-turn lane.

Combined left-turn / through / right-turn lane with opposing vehicular flows and with pedestrians

Since this lane arrangement is very complex, either the shared left-turn / through or shared right-turn / through procedure is applied depending on the more critical combination of the directional split, pedestrian flow and opposing vehicular traffic flow.

Shared lane with limited queueing or discharge space

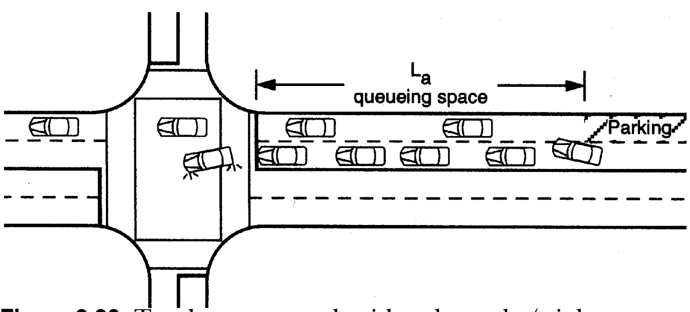


Figure 3.26 Two-lane approach with a through / right-turn and through / left-turn lanes, with limited queueing space.

The saturation flow values for the two lanes included in this situation may be approximated by a multiplication of adjustment factors from the procedure for the limited queueing space ("Queueing and discharge space" on page 3-36) and the procedures for shared lanes ("Protected left-turn and through movements" on page 3-44 for protected phases and "Permissive left-turn and through movements" on page 3-44 for permissive phases). The combined effect may, however, be significantly more severe and additional judgmental reductions may be needed. Although detailed calculation procedures for the determination of the saturation flow for the permissive phasing of shared lanes with short queueing space are available (Miller 1968, Richardson 1982, ITE 1984), they are complex. As illustrated in the example in *Figure 3.26*, the left-turning vehicles may block the whole intersection approach. A similar problem may be caused by right-turning flows.

Exclusive U-turn lane where left turns are not permitted

The saturation flow rate for a U-turn movement during a protected phase is significantly lower than the saturation flow rate for a projected left turn from an exclusive lane. However, calculating the saturation flow rate for a U-turn movement is complex since it is dependent on a number of factors including lane width (approach and discharge), number of discharge lanes, percent of heavy vehicles, median width, on-street parking, transit stops, pedestrian activity, etc. Ideally, the adjusted saturation flow rate is determined based on site specific surveys. In the absence of survey data, a reasonable estimate would be a saturation flow rate in the range of 1,000 to 2,000 pcu/h.

Special lanes

This category includes dedicated lanes for buses or streetcars, sometimes with special signal priority, or roadways where additional lane width has been provided for bicycles. Since operating conditions vary from location to location, surveys of the actual saturation flows are advisable.

If mixed use is allowed, such as for high occupancy vehicle (HOV) lanes with buses, taxis, and carpools, vehicle types should be converted to passenger car units. If only buses use a lane, no additional adjustment factor is usually necessary. The bus passenger car unit equivalent ("Flow in passenger car units per hour (pcu/h)" on page 3-15) is normally sufficiently representative of the conditions.

If the lanes are fully dedicated, the particular type of vehicle may be used as a unit of measurement, such as buses, trucks or bicycles. Buses or bicycles per hour (bus/h, bicycle/h) and buses or bicycles per hour green (bus/h, bicycle/h) are then used as the units of flow and saturation flow respectively. Measurements from Ottawa (Ottawa-Carleton 1994) indicate an average saturation flow of 865 *buses* per hour green (bus/h).

Bicycle arrival flows and bicycle saturation flows are discussed in "Pedestrians, Bicycles, and Transit" on page 3-67.

3.2.13 Saturation flow in planning applications

Because of the inherent uncertainty involved in planning applications, a lower degree of precision may be tolerated. Not all adjustment procedures may therefore be needed. The saturation flow values for appropriate approach environment may be used with only essential modifications. It is also not necessary to strictly follow the lane-by-lane saturation flow calculation.

Geometric conditions

The use of proper saturation flow values for the intersection environment will usually compensate for any deviation of the intersection geometry from ideal conditions. Specific adjustments are therefore normally not necessary.

Traffic and control conditions

Protected and permissive left turns as well as right turns with pedestrians have the most critical impact on saturation flow. Simplified saturation flow adjustment procedures for planning applications are outlined in the following sections. They should not be used for detailed signal analysis or design. In environments with high pedestrian traffic or very restricted geometry, such as in downtowns, the planning procedures may not be adequate and detailed adjustment factors should be determined.

Approaches with dedicated left-turn lanes

The approximate values listed in *Table 3.21* may be applied. Interpolation is possible.

The values in *Table 3.21* are based on the assumptions of the opposing flow using two to three lanes and the subject green interval constituting about 40% of the cycle. If the conditions are significantly different, a detailed analysis using the procedure described in "Permissive left turns in exclusive lane without pedestrian flow" on page 3-42 is preferable, because the left-turn movements are often critical, even at the planning stage.

Table 3.21 Saturation flow values for left-turn lanes in planning applications

Left-turn flow (pcu/h)	Phase	Opposing flow[1,2] (pcu/h)	Adjusted left-turn saturation flow (pcu/h)
regardless	protected	not applicable	S basic
< 100	permissive	in all cases	-[3]
≥ 100	permissive	100	1500
		200	1100
		400	700
		800	450
		1200	200
		1600	0

1. The opposing flow per hour q'_o is used, *not the rate* of opposing flow per hour green.
2. The number of opposing lanes should also be considered. This impact is shown in *Table 3.16*.
3. Left-turn arrival flow may be ignored.

Approaches without dedicated left-turn lanes

For approaches without dedicated left-turn lanes, individual intersection movements must be allocated to lanes as described in "Permissive left-turn and through movements" on page 3-44 and "Other left-turn situations" on page 3-45.

Right turns

For right-turn saturation flow, the same values as for straight-through lanes may be generally assumed. However, counts completed in the Region of Waterloo (1999-2001) show values of 1,425 to 1,450 for right turn lanes. Similarly, saturation flows for a shared right-turn and through lanes are considered equal to the saturation flow values for a straight-through lane. Saturation flows for each lane of a dual right-turn are assumed to be equal. In all of these configurations, if significant pedestrian activity exists, procedures of "Right turns in exclusive lanes" on page 3-46 apply.

Right turns on red (RTOR) are usually ignored at the planning level.

3.2.14 Saturation flow surveys

Since the accuracy of saturation flow estimates significantly influences the quality of analysis or design, the saturation values for specific local conditions should be determined from reliable field measurements wherever possible. Chapter 6 describes the procedures for the surveys and the analysis of their results.

3.3 Timing Considerations

In addition to the assessment of arrival flows and saturation flows, timing requirements must be considered as the third essential component in both design and evaluation. The existing or future requirements and constraints of the control system must be identified and taken into consideration.

This Section describes the principal features of intersection signal control operations. Chapter 4 *"Planning and Design"* identifies the calculation procedures for the cycle time and the method of allocating green intervals within a given cycle. Section 3.5.2 "Traffic adaptive control" includes information relevant to traffic actuated operation. Pedestrian timing requirements are described in Section *3.4.1*

This Section focuses on the principles and constraints of vehicular timing requirements common to the design and evaluation tasks. The following timing parameters are included:

- phase composition
- cycle structure
- green intervals
- red intervals
- amber intervals
- all-red periods

Although all of these control parameters are known for existing intersections, it is useful to review, assess, and if necessary, correct them before proceeding to the evaluation stage. For new intersections, these features must be considered early in the analytical stages because they are needed for the determination of lane arrival flows and saturation flows.

3.3.1 Phase composition and cycle structure

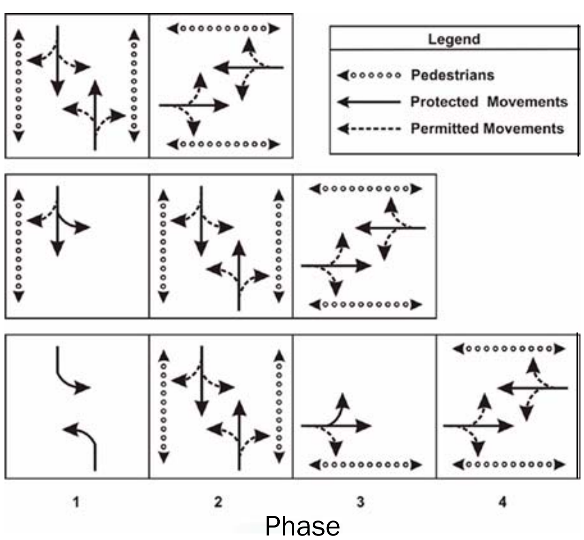


Figure 3.27 Typical phase compositions and cycle structures.

Phase composition is the grouping of intersection directional movements into phases. It also includes the allocation of these movements to individual approach lanes. The number of phases and their sequence is referred to as *cycle structure*.

Cycle structure and the composition of individual phases depend on geometric constraints, and traffic flow distributions, as well as some strategic network considerations. The analysis of control operations at existing intersections should include a review of these aspects.

For intersection control systems under design or in the planning stages, it is useful to devise phases which address more than one critical movement. Nevertheless, it may be practical to start with a simple composition of phases and cycle structure because the evaluation stage (Section 4.6 "Evaluation" on page 4-95) indicates which movements are the source of specific problems. The accompanying computer program, InterCalc, can also analyze feasible combinations of movements and suggest phase structures. The signal control hardware and associated software that are used may pose some restrictions.

The following points are general guidelines:

- examine weekly, daily and hourly traffic variations, and define typical flows for each major design period;
- examine turning movements, especially the left turns. Heavy flows may require protected phases;
- a high degree of variation among daily or weekly flow patterns, or short-term flow fluctuations in some of the movements may require different phase compositions and cycle structures during various times of the day or days of the week;
- very short term fluctuations of arrival flows may lead to modes of operation different from the usual fixed time control. Control systems with special decision logic or traditional forms of traffic actuated operation may be considered for various periods if the control hardware and software are not a constraint;
- from a capacity and delay point of view, those phase compositions and cycle structures that feature minimum lost time ("Lost time" on page 3-64) or fewer phase may be preferable;
- safety concerns may also indicate a need for a specific timing scheme;
- since directional lane markings that designate the allocation of intersection movements to individual lanes cannot be changed with the change of the control mode and phase composition, a common lane marking scheme may pose a limitation for some of the analysis periods. Variable traffic control signs, however, may reduce this problem;
- special measures regarding pedestrian traffic may be required for some operating schemes;
- restrictions of certain vehicular or pedestrian movements may be required as a result of a combination of geometric conditions and competing vehicular or pedestrian requirements;
- the impact of the mode of operation, phase composition and cycle structure on the adjacent network and on transit operations should be addressed.

An appropriate cycle structure and a logical composition of the phases are the central features of the timing scheme. Typical combinations of intersection movements and examples of possible corresponding cycle structures are shown in Figure 3.27 on page 3-55. Table 3.22 on page 3-57 lists general indicators that may lead to the introduction of a protected left-turn phase. A more rigorous protected left turn warrant can be found in Form B4-1 from the Manual of Uniform Traffic Control Devices for Canada (MUTCDC-TAC 1998).

Table 3.22 Protected left-turn phase indicators[1]

Criterion	Conditions
Demand	A moderate to heavy left turn volume is present throughout the peak hour.
Transit	There are public transit or railway vehicles operating in the median, which is parallel to the left-turn lane.
Geometry	Left-turns are permitted from two lanes on one approach where there is an opposing through movement.
Safety considerations	Drivers have difficulty making left-turns safely throughout the peak hour.
Queue reach	The left-turn queue frequency extends beyond the left-turn lane, thereby blocking the through movement.
Visibility / Sightline	Intersection geometry creates a visibility problem which may be alleviated by a left-turn phase (should be fully protected).
Speed of opposing traffic	The speed of approaching traffic is sufficiently high to make driver judgement of gaps difficult.

1. **Source:** Adjusted from MUTCDC (TAC 1998)

Left -turn phasing – protected/permissive versus fully protected

Once it has been established that a left turn phase is required, it is necessary to assess the type of left turn operation that should be implemented: permissive/protected or fully protected. The following points should be considered:

- The permissive/protected type of operation is the simplest and makes the most efficient use of intersection capacity. This type of operations should be considered if the capacity analysis indicates that a single left turn lane is sufficient, there are no special geometric or visibility issues, and there are no other unique safety concerns;
- A fully protected left turn phase should be considered when geometric or visibility problems exist at the intersection;
- A fully protected left turn phase should be considered where a capacity analysis indicates that dual left turn lanes are required. Some jurisdictions permit permitted/protected dual left turn lanes in situations where the geometry of the intersection and approaches allows for proper turning treatment and opposing traffic is such that motorists will not have issues judging gaps.

Cycle time

The calculation procedures to determine the minimum and optimum cycle times for vehicular and pedestrian traffic are described in “Cycle time” on page 4-86. Other pedestrian timing requirements are included in “Pedestrians, Bicycles, and Transit” on page 3-67.

3.3.2 Green interval

The focus of the following sections is on the constraints and principal considerations regarding green intervals. The procedure for their determination is described in "Green allocation" on page 4-88. The total available green time in the cycle is normally allocated to individual phases.

Minimum green interval

Drivers do not expect an immediate termination of a signal indication that has just started. *Table 3.23* lists empirically established minimum signal intervals or periods. They represent a general guide since some local practices in Canada may be more specific.

Table 3.23 Minimum signal timing intervals[1]

Interval	Desirable minimum(s)	Acceptable minimum(s)[2]
Circular green for roads posted at less than 80 km/h	10.0	7.0
Circular green for roads posted at 80 km/h or more	20.0 (Main Road) 10.0 (Side Road)	15.0 (Main Road) 7.0 (Side Road)
Advanced green	7.0	5.0
Flashing advanced green clearance	2.0	1.5
Circular amber	3.0	3.0
Amber arrow	3.0	2.0[3]
All red	1.0	1.0
Transit priority	5.0	3.0
Pedestrian walk	7.0	5.0
Pedestrian clearance	5.0	3.0

1. **Source**: Ontario 2001
2. Acceptable minimums may be implemented during emergency vehicle pre-emption or during low volume time periods (e.g. overnight).
3. NEMA controllers are limited to a minimum of 2.7 seconds

Green interval and effective green interval

Figure 3.5 on page 3-24 and Figure 3.6 on page 3-25 illustrate the concept of saturation flow and show that the highest rate of flow does not start immediately after the beginning of the green interval. The initial vehicular headways are significantly longer and stabilize at a saturation flow headway only after the sixth or seventh vehicle, counting those that have waited in the queue at the beginning of the green interval. *Figure 3.5* also shows that this highest flow rate does not cease immediately after the amber signal has been displayed because drivers close to the stop line at that moment cannot stop immediately.

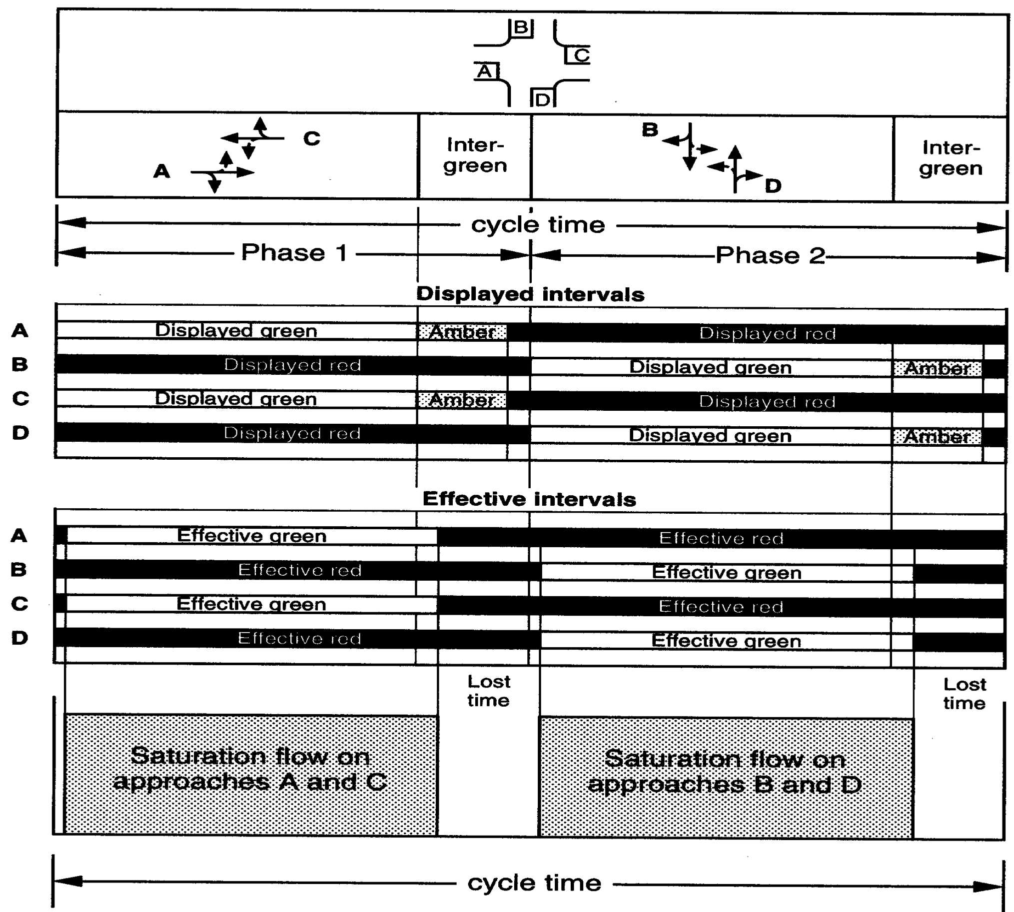


Figure 3.28 Basic signal timing parameters consistent with the effective green interval and saturation flow representation.

The result is a shift in the utilization of the green interval as illustrated in the upper part of *Figure 3.28*. The cumulative value of saturation flow and its adjustment for the duration of the green interval take into account much of the initial time loss. As a result, the net value of time actually used by the vehicles is slightly longer than the displayed green interval. This is called *the effective green interval*, and in typical instances has been measured as one second longer than the displayed green interval. Therefore:

$$g_e = g + 1$$

where:

g_e = effective green interval (s)

g = displayed green interval (s)

Under conditions close to or at capacity, the effective green interval may be longer by two or more seconds than the displayed green interval. These values are not recommended for design but may be important in the evaluation of existing signal operations.

Critical lanes

The analysis, design and evaluation of signalized intersections, including most planning tasks, proceed on a lane-by-lane basis. Not all the lanes, however, are equally important. Normally, in every phase there is only one lane for which the relationship between the arrival flow and saturation flow results in the longest green interval requirement. Such lanes are called *critical lanes*. The number of critical lanes equals the number of phases in a cycle and, together, they have a decisive influence on the cycle time.

A critical lane can be recognized by the *highest flow ratio* in a given phase ("Lane flow ratio" on page 4-85 and "Intersection flow ratio" on page 4-85):

$$y_{crit\,j} = \max y_{ij} = \max (q_{ij} / S_{ij})$$

where:

$y_{crit\,j}$ = flow ratio for the critical lane in phase j

y_{ij} = flow ratio for lane i in phase j

q_{ij} = arrival flow in lane i discharging in phase j (pcu/h)

S_{ij} = saturation flow in lane i discharging in phase j (pcu/h).

Normally, most intersection movements take place only during one phase. The above equation, however, also applies to movements from lanes that discharge their arrival flows during two or, exceptionally, more phases. For example, a left-turn lane may discharge its flow in a leading protected phase and in the following permissive phase.

The sum of the flow ratios for the critical lanes is called the *intersection flow ratio* and provides an indication of the quality of service at that location.

3.3.3 Intergreen period

The intergreen period, known as the signal change interval in the USA, is defined as the time between the end of the green interval for one phase and the beginning of the earliest green interval for the next phase in the same signal cycle. It usually consists of an amber interval and an all-red period separating potentially conflicting movements.

Amber interval

A review of current North American practices is included in a report by the ITE Technical Council Task Force 4TF-1, Determining Vehicle Signal Change and Clearance Intervals (ITE 1994).

Most Canadian jurisdictions apply amber intervals consistent with the Canadian Model Rules of the Road (TAC 1996) and the definition in the proposed Part B of the Manual of Uniform Traffic Control Devices for Canada (TAC 1998). This definition requires drivers facing an amber indication (both circular and amber arrow signals) to stop before crossing the stop line or before entering the crosswalk on the near side of the intersection, unless such stop cannot be made safely. Some jurisdictions, however, may use a different definition. Local laws and rules govern these considerations.

Driver behaviour after the appearance of the amber signal is very complex. Local habits, different amber duration and display practices, different legal rules and their enforcement levels, and other factors influence amber utilization and make information transfer difficult.

Some recent research also suggests that amber utilization is largely independent of approach speeds (ITE 1994).

The calculation of amber intervals usually follows a formula based on the kinematic principles of uniformly decelerated motion. It provides sufficient time for a reasonably alert driver in the decision zone at the onset of the amber interval, to initiate the stopping action and stop within the stopping distance. This time would allow stopping the vehicle at the stop line at a comfortable average deceleration rate under somewhat less than ideal roadway conditions.

$$A = t_{pr} + v / (2a + 2gG)$$

where:

A = amber interval (s)

t_{pr} = time of perception and reaction (s)

v = speed (m/s)

a = average deceleration rate (m/s^2) usually taken to be 3.0 m/s^2

g = gravitational constant (9.81 m/s^2)

G = average grade of approach within 50 m of the stop line (%/100); downhill grade is taken as negative.

Specific values for the parameters of the formula vary for different regions. The Ontario Traffic Manual - Book 12 (Ontario 2001) applies amber intervals shown in *Table 3.24*. Note that Ontario uses 1.8s perception reaction time (t_{pr}) for posted speeds equal to or higher than 80 km/h and 1.0s for a posted speed lower than 80 km/h.

Table 3.24 Amber intervals at level approaches used in Ontario[1]

Posted Speed (km/h)	40	50	60	70	80	90	100	110
Amber clearance for 1.0 seconds perception + reaction time(s)	3.0	3.3	3.7	4.2	4.6	5.1	5.5	6.0
Amber clearance for 1.8 seconds perception + reaction time(s)	3.6	4.1	4.5	5.0	5.4	5.9	6.3	6.8

1. **Source:** Ontario 2001

Some jurisdictions use the operating speed to determine the amber interval, while others use the posted speed. This is represented in *Table 3.25* by the values used by the City of Edmonton. The higher value of either the speed limit or operating speed is usually considered.

Table 3.25 Amber intervals at level approaches used in Edmonton

Speed limit[1] (km/h)	30	35	40	45	50	60	70	80	90
Amber interval (s)	3.0	3.0	3.0	3.0	3.0 or 4.0 [2]	4.0	4.0 or 5.0 [3]	5.0[3]	n/a

1. If operating speed deviates significantly from the posted speed limit, operating speed is used as the basis for the calculation
2. 4.0 s for intersections of roadways with four or more lanes, or where operating speed exceeds 50 km/h speed limit
3. 5.0 for approaches with a long distance from an upstream intersection. In some instances, advanced flashing amber warning signals are considered.

Source: Edmonton 1991

The ITE report (ITE 1994) defines speed used in the calculation of the amber interval as the speed limit or 85% percentile speed, whichever is higher. *Table 3.26* includes amber intervals interpolated for speeds in 10 km/h increments from the full ITE report.

Table 3.26 Amber intervals from the ITE 1994 report

	Speed (km/h)				
	50	60	70	80	90
Approach grade (%)	Amber (s)[1]				
Level 0	3.3	3.8	4.2	4.9	5.1
Uphill +1	3.2	3.7	4.1	4.8	5.0
Uphill +2	3.2	3.6	4.0	4.7	4.9
Uphill +3	3.1	3.5	3.9	4.6	4.8
Uphill +4	3.0	3.4	3.8	4.5	4.6
Downhill -1	3.3	3.7	4.3	5.1	5.3
Downhill -2	3.4	3.9	4.4	5.2	5.4
Downhill -3	3.5	4.0	4.5	5.3	5.6
Downhill -4	3.6	4.1	4.7	5.5	5.7

1. The ITE report advises caution in the use of amber intervals longer than 5.0 s that seem to encourage more aggressive drivers to disrespect the signal indication, the speed limit, or both.

Source: adapted from ITE 1994

All-red period

The all-red period is the time when all intersection signals display red indications. The vehicular or pedestrian signals for movements that continue in the following phase, and are not in conflict with the movements of the ending or the starting phase, are an exception and may display different signal indications. The all-red period fills up the difference between the amber interval and the required intergreen period. It cannot be negative. The ITE report (ITE 1994) and some Canadian local practices suggest a minimum 1.0 s for phases that carry straight-through movements.

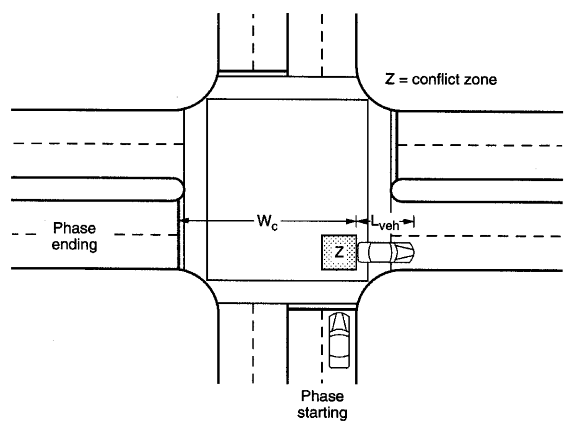


Figure 3.29 Variables for the determination of the intergreen period.

The determination of the all-red period is based on the conditions shown in *Figure 3.29* and the following equations:

$$r_{all} = I - A$$

$$I = i + (W_c + L_{veh}) / v_c$$

where:

r_{all} = all-red period (s)

I = intergreen period (s)

i = amber overrun (s), representing the period from the beginning of the amber interval to the point in time when the last vehicle crosses the stop line. Depending on local driver behaviour, it may be taken as the amber interval (i = A), or shorter by 1.0 s or less than the amber interval (i = A-1.0). A definition of the "last" vehicle may be based on the 85th to 95th percentiles of vehicles *legally* utilizing the amber interval.

W_c = distance to clear (m), measured from the stop line to the far end of the potential conflict zone for the most critical combination of lanes for which the green interval terminates, and lanes for which the green indication is about to start.

L_{veh} = length of the clearing vehicle (m), usually taken as the space for a passenger car, because longer vehicles act as barriers; normally, L_{veh} = 6.0 m

v_c = clearing speed of the last vehicle (m/s). Local practices in Canada vary from 7.0 m/s to the speed limit expressed in m/s.

Where the starting phase features long approach distances, the time required for the first vehicle to reach the near side of the conflict area from the stop line is sometimes subtracted. The set of assumptions includes a "flying" start and crossing the stop line at the very first moment of the green interval.

The requirements of a situation where vehicles are clearing the intersection during the intergreen period and pedestrians will be entering the conflicting crosswalk should also be verified, where applicable. In this case, the formula for the intergreen period is:

$$I = W_c / v_c$$

where:

W_c = distance from the stop line to the far end of the crosswalk (m)

v_c = clearing speed of the last vehicle (m/s).

The length of the vehicle need not be considered, because after the vehicle of the ending phase has entered the crosswalk, it acts as a barrier for pedestrians.

The longest intergreen period, determined for all combinations of starting and clearing lanes and pedestrian movements between two consecutive phases, governs the all-red period.

All-red period for left turns

The intergreen period of the advanced protected left-turn phase followed by a permissive left-turn phase does not normally require a displayed all-red period. Drivers are preparing for the left turn and are usually aware of the opposing traffic that will obtain the right-of-way. Because they will be turning, the approach speeds are usually lower.

An all-red period, however, is required following the leading protected simultaneous left-turn phase.

The Manual of Uniform Traffic Control Devices for Canada (TAC 1993a) illustrates these and several other possible sequences for left turns and should be followed.

3.3.4 Lost time

The purpose of the intergreen period is to separate the conflicting movements of two consecutive phases in time. By definition, some of this time is lost for the movements of traffic. The concept is shown in Figure 3.28 on page 3-59.

Phase and lane lost time

Consistent with the calculation of the effective green interval, the *lost time* in phase j that is followed by phase (j+1) (Figure 3.28 on page 3-59) is determined as:

$$l_j = g_j + I_j - g_{ej}$$

where:

l_j = lost time of phase j (s)

g_j = longest displayed green interval of phase j (s)

I_j = intergreen period between phases j and j+1 (s)

g_{ej} = effective green interval of phase j corresponding to the longest displayed green interval.

The following condition must be met:

$$c = \Sigma_j g_{ej} + \Sigma_j l_j = \Sigma_j g_j + \Sigma_j I_j$$

where:

c = cycle time (s).

Given the saturation flow concept described in "The concept of saturation flow" on page 3-23, the typical effective green interval in Canadian urban areas has been identified as being 1.0 s longer than the displayed green interval. The lost time between two consecutive phases is therefore

$$l_j = I_j - 1.0$$

where:

l_j = lost time of phase j (s)

I_j = intergreen period between phases j and j+1 (s).

Where a local value of the effective green interval has been determined by surveys, the magnitude of the lost time must be consistent with this value. For phase sequences that allow a lane movement to continue, the lost time may differ from the above equation but must also correspond to the effective green interval used.

Although the concept of lost time appears simple, it is a complex issue. Local observations, especially regarding protected / permissive or permissive / protected phase sequences are highly recommended.

Intersection lost time

When all of the critical lanes operate at capacity, the effective green intervals are used by their respective saturation flows. The remaining time in each phase is lost for the movement of traffic. This is the condition of a fully utilized two-phase signal cycle shown in Figure 3.28 on page 3-59. The sum of the lost times for all phases in the cycle is the intersection lost time, and is determined as:

$$L = \Sigma\, l_j$$

where:

L = intersection lost time (s)

l_j = lost time associated with phase j.

3.3.5 Other timing constraints and issues

Constraints regarding the composition of individual phases and cycle structure, as well as individual signal intervals, may include but are not restricted to the following additional considerations:

- pedestrians
- cyclists
- transit operations
- signal coordination
- special control requirements, such as traffic actuated phases or transit priority signals
- queue control
- demand management
- control equipment
- control software.

Pedestrian Scramble

During a pedestrian scramble, all vehicles are held to allow pedestrians to cross in all directions. Exclusive pedestrian phases are normally required only where the volumes of crossing pedestrians are extremely high and safety is impaired by the use of normal display intervals which parallel the (vehicles) signal head operations[1]. Implementing a pedestrian scramble requires the estimation of the time needed for this special phase, and consideration of delay to pedestrians as well as overall intersection operations. Some of the associated advantages and disadvantages include:

Advantages:

- Eliminates conflicts between pedestrians and turning vehicles
- Allows pedestrians to cross intersection in any direction (including diagonally)
- potentially reduces delays to motorists in curb lanes

Disadvantages

- Increases delay for all pedestrians
- Increases potential for pedestrians to disobey signals
- Increases delay for right turns since right turns on red cannot be permitted

1.**Source**: OTM Book 12 (Ontario 2001)

- Disruptive to signal co-ordination since longer cycle lengths and shorter green times are normally required
- Increases queue lengths
- Potentially exceed storage space available for pedestrians

3.4 Pedestrians, Bicycles, and Transit

Pedestrians, cyclists and transit vehicles are important components of intersection operation. They require the same attention as the other vehicular users of the intersection. With the increasing drive towards sustainability, accommodating these users is becoming more important, particularly in larger municipalities.

Pedestrians and cyclists are more vulnerable than motorized vehicles. The ranges and distributions of their characteristics, such as speed or the degree of compliance with the rules of the road, are usually very broad. Older people and parents with children may move much slower than teenagers and the middle-aged. Cyclists range from slow moving recreational riders to aggressive couriers.

Buses, articulated buses, trolley buses and streetcars generally exhibit the same operational properties as other motorized vehicles. Larger, heavier streetcars have a slower starting speed. As a result, the evaluation criteria may carry greater weight for the lanes or directions used by public transit. For that reason, special provisions for transit vehicles are often included in the design of the intersection geometric and control systems.

Pedestrian, bicycle, and transit vehicle flows, together with special geometric and control features provided for them, may significantly affect the overall quality of service provided by the intersection. It is often necessary to assess the operational features separately for these groups of users.

3.4.1 Pedestrians

The following descriptions are based on the definitions in the TAC Manual of Uniform Traffic Control Devices. Since individual jurisdictions may apply somewhat different interpretations, local legal and behavioural considerations govern and should be consulted.

Pedestrian walk interval

Pedestrians facing the walk signal indication (walking pedestrian symbol) may enter the roadway and proceed in the direction of the signal. The visual signal may be supplemented by an audible signal (TAC 1993b). The duration of this interval depends on whether the crosswalk offers an intermediate place (i.e. median) to wait for the walk signal in the following cycle. Figure 3.30 on page 3-68 illustrates two typical geometric situations.

Crosswalks without median refuge

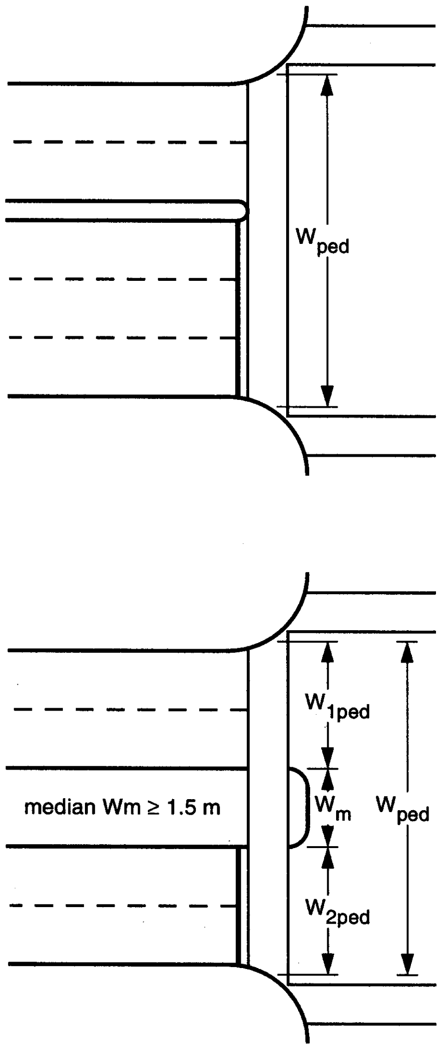


Figure 3.30 Crosswalk configurations for walk interval and pedestrian clearance considerations.

These crosswalks do not feature a sufficiently wide median or an island on which the pedestrians can wait. The minimum duration of the walk interval depends primarily on the perception and other psychological factors related to the width of the roadway or length of the crosswalk and, to a lesser degree, on pedestrian speed. The minimum pedestrian walk interval should allow time for the pedestrians to notice the change of the signal indication and to cover a sufficient distance into the crosswalk. It is usually defined as

w_{min} = 10 s (exceptionally 7 s)

The exceptional value should only be used if the right-turn or left-turn saturation flow and pedestrian traffic are seriously affected by each other ("Permissive left turns in exclusive lane with pedestrians" on page 3-44 and "Right turns in exclusive lanes" on page 3-46), where public acceptance can be anticipated and where the objectives of the intersection operation permit it.

On the other hand, longer minimum walk intervals may be desirable in areas with high pedestrian flows, such as central business districts, in order to accommodate all pedestrians waiting to cross. This operational objective may, however, be in conflict with the conditions stated in the previous paragraph.

Longer walk intervals may also be used at intersections with low pedestrian traffic. The few pedestrians arriving at the crosswalk usually do not impede the vehicular flows and should not therefore be delayed without reason.

Crosswalks with median refuge

A crosswalk with a very wide (7 m or more) centre island, or a median that practically divides it into two independent sections, may be treated as two separate crosswalks. To this end, the pedestrian signal heads and actuation devices must be located on both the sidewalks and the island or median. The crossing times should allow pedestrians to comfortably cross the longest crossing to the refuge median. Signage should also be used to identify to pedestrians that a two-stage crossing is required.

For single crosswalks with a refuge (a centre island or a median wider than 1.5 m), yet not wide enough for a complete separation of both crosswalk portions (less than 7.0m), the first group of pedestrians should be able comfortably to reach a point beyond the refuge at the end of the walk interval. The timing should assume that pedestrians cross the longest crossing to the refuge island. If this condition is not met, it may be difficult for some pedestrians to decide whether to stop at the refuge or to continue walking. It is the practice of some jurisdictions not to cater to providing enough “Walk” time to clear the centre median due to implications on cycle lengths; however, this may be mitigated by providing pedestrian countdown timers (PCS), or providing multi-stage pedestrian crossings.

At traffic actuated intersections where a central refuge is provided and used in the calculation, a pedestrian push-button must be located in the refuge to prevent entrapment of a pedestrian.

As illustrated in Figure 3.30 on page 3-68 the minimum walk interval is calculated as:

$$w_{min} = [\max(W_{1ped}, W_{2ped}) + W_m + W_3] / v_{ped}$$

where:

w_{min} = minimum walk interval (s)

$\max(W_{1ped}, W_{2ped})$ = the longer portion of the crosswalk or the width of the wider roadway section. It is defined as the distance measured mid-way between the lines that designate the crosswalk (m).

W_m = width of the median (m)

W_3 = additional width provided to minimize the potential that a pedestrian would unnecessarily stay in the median, taken as one lane, i.e., 3.0 to 3.5 m

v_{ped} = walking speed adequate for the design conditions (m/s). Under normal circumstances, 1.2 m/s or 1.0 m/s is practical (TAC 1993b, TRB 1994, Coffin and Morrall 1995, Toronto 1995b)

The distinction between the case of a crosswalk with and a crosswalk without a refuge depends on regional practices. Many Canadian jurisdictions apply the procedure for crosswalks without a refuge, even where a median is available.

If the median is not intended for pedestrian refuge it should not be designed to extend into the crosswalk.

Pedestrian clearance period

The pedestrian clearance period provides a reasonable time for the pedestrian who entered the crosswalk at the very last moment of the walk interval to reach a designated pedestrian refuge. Alternatively, the pedestrian should be able to reach the opposite side of the roadway at a comfortable speed before the conflicting green interval commences. The minimum pedestrian clearance period generally consists of a flashing hand signal indication plus a steady hand indication.

Pedestrians facing the *flashing* hand signal indication must not enter the roadway in the direction of the signal. A pedestrian who has begun crossing while facing the walk signal indication, however, may complete the crossing to the designated refuge area. Pedestrians facing the *steady* hand signal indication must not enter the roadway in the direction of the signal (TAC 1993a).

The pedestrian clearance period may be determined as follows:

$w_{clear} = w_{ped} / v_{ped}$

where:

w_{clear} = pedestrian clearance interval (s)

w_{ped} = length of the crosswalk or the width of the roadway to the nearest refuge measured mid-way between the lines designating the crosswalk (m). Where the crosswalk has a central refuge (*Figure 3.30*), the longer section governs.

v_{ped} = walking speed adequate for the design conditions (m/s). A value of 1.2 m/s or 1.0 m/s is usually applied although lower pedestrian speeds may be appropriate in special circumstances. In industrial areas with few pedestrians, 1.5 m/s is sometimes used (TRB 1994, Coffin and Morrall 1995, Toronto 1995b).

Pedestrian Countdown Signals (PCS)

A pedestrian countdown signal integrates a countdown timer into the traffic control signal to show pedestrians how many seconds are remaining until the vehicular traffic will be allowed to proceed through the crosswalk.

National guidelines for the optional use of PCSs have been prepared by the Transportation Association of Canada's (TAC) Traffic Operations and Management Standing Committee. The guidelines provide recommendations on their installation, application, configuration, and operation. The guidelines also identify conditions in which PCS may be beneficial or detrimental.

General benefits include:

- Increased pedestrian understanding compared to the conventional flashing hand "don't walk" display
- Beneficial in areas with high percentages of mobility-challenged pedestrians such as seniors and children

General disadvantages include:

- Providing an accurate countdown for actuated phases
- Potential increase in crashes or conflicts associated with vehicles changing their behaviour based on the countdown (e.g. racing the phase termination).

3.4.2 Bicycles

The method of accommodation of bicycle flows depends on whether they use the standard intersection approach lanes, wider intersection approach lanes, special bicycle lanes or bikeways.

Bicycle flow in mixed traffic

Bicycle flows in mixed traffic are converted to passenger car units and included in the flow for the appropriate lanes. Their passenger car unit equivalent may vary from negligible for random cyclists in wide lanes to more than 1.0 where individual cyclists in a narrow lane slow down the following vehicular traffic that has no opportunity to pass. Typically, however, where groups of cyclists form during the red interval in a standard intersection lane, their passenger car unit equivalent is about 0.2 (See "Saturation flow in pcu/h" on page 3-25 and Table 3.2 "Passenger car unit equivalents" on page 3-15).

Normally, cyclists use only the extreme right lane for straight-through and right-turn movements, and the left lane for left-turn movement. At complex intersections without special bicycle facilities, some cyclists prefer to dismount and use the crosswalks.

Bicycle flow in wider lanes

Bicycle flows in wider lanes (usually the right lane) may be considered separately on the additional width, with bicycles per hour used as the units of traffic flow.

Bicycle flow on shoulder, in separate lane or on bikeway

These flows are considered separately with bicycles per hour used as the units of traffic flow .

Intergreen periods for bicycles

Practical experience indicates that amber intervals and all-red periods adequate for motorists are also adequate for cyclists (Wachtel et al 1995). Under special circumstances, different all-red timing may be considered.

3.4.3 Transit vehicles

General remarks regarding the system aspects of intersection signal operations, including transit, are discussed in "How to Use this Guide" on page 2-7. Person flows and vehicle occupancy are described in "Person flow and vehicle occupancy" on page 3-16; the influence of streetcar tracks in the pavement surface on vehicular saturation flow in "Pavement conditions" on page 3-27; the influence of transit stops on saturation flow in "Transit stops" on page 3-37; bus saturation flow in dedicated lanes in "Special lanes" on page 3-52; special transit timing considerations in this chapter; and the use of total overall person delay as an evaluation criterion in "Non-vehicular delay" on page 4-105.

Streetcars operating in mixed traffic in the centre lanes of a street require special consideration. Where no streetcar loading platform exists adjacent to the streetcar lanes, streetcars influence the operation of the lane with the rails and lane(s) between the centre lane and the curb. It is therefore logical to apply saturation flow adjustments due to transit stops to all lanes affected by the streetcars. Streetcars have been implemented in more cities in recent years, with a range of operating strategies from mixed traffic to exclusive lanes. The Cities of Toronto, Ottawa, Calgary and Edmonton are among the Canadian cities which have existing streetcar or light rail transit systems, and they may have additional information to share regarding the effect of streetcars on traffic operations.

3.4.4 Transit signal priority

Primer on TSP

Transit Signal Priority (TSP) is a control strategy that provides preferential treatment to surface transit vehicles (buses and streetcars) operating in mixed traffic along urban corridors. The main objective of TSP strategies is to reduce transit vehicle delays at signalized intersections through the modification of signal time settings, thus improving transit operational efficiency and level of service. TSP treatments can be classified into three types, which also roughly represent the evolution of TSP and its level of sophistication over time. These types are described briefly below.

Passive TSP

Under Passive TSP, signal timing plans are designed off-line based on transit vehicle frequency and speed. The timing plans are then deployed at the corresponding intersections, where they are executed continuously without regard to the presence of transit vehicles. As such, no vehicle detection technology is required for Passive TSP, which reduces the cost involved. Passive TSP is most effective under conditions of high transit vehicle volumes. However, they may incur unnecessary delays to cross-street traffic, if transit vehicle arrivals are not highly regular and predictable (which is often the case). Passive priority may include one or more of the following treatments: (i) signal coordination based on transit travel times, (ii) phase splitting, and (iii) cycle length adjustment. Studies have shown that this is not a very effective way to provide priority to transit vehicles in the traffic stream.

Active TSP

Under this scheme, priority is only granted when transit vehicles are approaching intersections, and as such, a technology for selectively detecting transit vehicles and communicating this information to the signal controller is necessary. Extension of the priority phase (i.e. green extension), early truncation of the non-priority phase (i.e. red truncation), and implementation of a transit-exclusive phase, are common strategies of active transit signal priority. Phase omission and rotation are also sometimes used, though there is a perception among many traffic engineers that this creates confusion among motorists.

There are two operational concepts for Active TSP. The first, Unconditional TSP, grants priority to any transit vehicle once it is detected upstream of the intersection. The priority is provided typically via green extension or red truncation, with offset transitioning (also known as offset recovery) implemented after the transit vehicle clears the intersection in order to recover signal coordination and to compensate the non-priority phases. Unconditional TSP has been successful in speeding up transit vehicles along arterial corridors. However, in some instances, transit vehicles may be granted priority when not needed (e.g. vehicle is ahead of schedule, or carrying few passengers), incurring significant delays to non-priority traffic (e.g. cross traffic).

The second type, Conditional TSP, grants priority selectively to transit vehicles that meet certain conditions based on deviation of vehicle from the schedule, or time elapsed since last awarded priority. The possibility of granting priority based on some threshold number of passengers on board the vehicle has also been discussed, but has not been applied in the North American context, due to the lack of accurate real-time passenger load counting systems.

Conditional TSP requires, in addition to the vehicle detection system, other systems or mechanisms for measuring whether the approaching vehicle meets the criteria for granting priority. These may involve an AVL (Automated Vehicle Location) system for measuring schedule adherence, and possibly in the future, APC (Automatic Passenger Counting) systems. Conditional TSP has the potential of limiting buses running ahead of schedule and of mitigating the impacts of Unconditional TSP on non-priority traffic. However, it also limits the absolute travel time benefit that might be achieved in the corridor.

Adaptive TSP

Adaptive TSP refers to a relatively new generation of priority schemes, which attempt to achieve advanced operational objectives by means of adaptive signal control. Examples of operational objectives include reducing total vehicle delay in the corridor and maximizing person throughput. Under Adaptive TSP, the traffic signal controller adapts its plan dynamically according to the criteria reflecting the desired objective. Adaptive signal control is increasingly common in Europe but has not been widely deployed yet in North America to date, but offers considerable promise for maximizing benefits for both transit vehicles and the general traffic.

Overview of the Canadian State of the Practice for TSP

Transit Signal Priority in Toronto

The Toronto Transit Commission (TTC) was one of the first transit systems to explore the use of TSP. After a first study that assessed the potential application of passive TSP, a demonstration was conducted in 1990, involving six intersections. The findings of this first demonstration were that delay reductions of 5 to 9 seconds at each intersection were attained, contributing up to a 20% reduction in total transit travel time. Other traffic was not significantly affected. Following this success, an incremental program was undertaken to equip over 150 intersections on seven streetcar routes. This resulted in the need for 10 fewer streetcars, and saved over $1 million a year in operating costs. The payback period was less than 5 years.

In 1997, a demonstration of bus TSP was undertaken, and the results were: bus delay decreased up to 46 percent, auto delay decreased marginally, and cross street traffic was not significantly affected. The detection system, however, caused some reliability problems. Since then, a program has equipped over 110 intersections on bus routes. The program is currently under evaluation to assess further deployment.

TSP strategies are implemented in several corridors in Toronto such as the King and St. Clair Streetcar routes. All TSP implementations are of the “Unconditional” type. They work mostly as follows. At an intersection with transit signal priority, if a streetcar has been detected at the upstream 'request' loop (approximately 100 m from the stop line), and has not yet crossed the 'cancel' loop at the stop line, the controller considers the “zone” to be “active” for this transit route direction. Two basic algorithms are used to provide signal priority for transit vehicles: transit-corridor green extension, and cross-street green truncation. In case of any disruption to the offsets (in reference to the master system clock) by the provision of signal priority an offset recovery routine is initiated. (See Figure 3.31 on page 3-74)

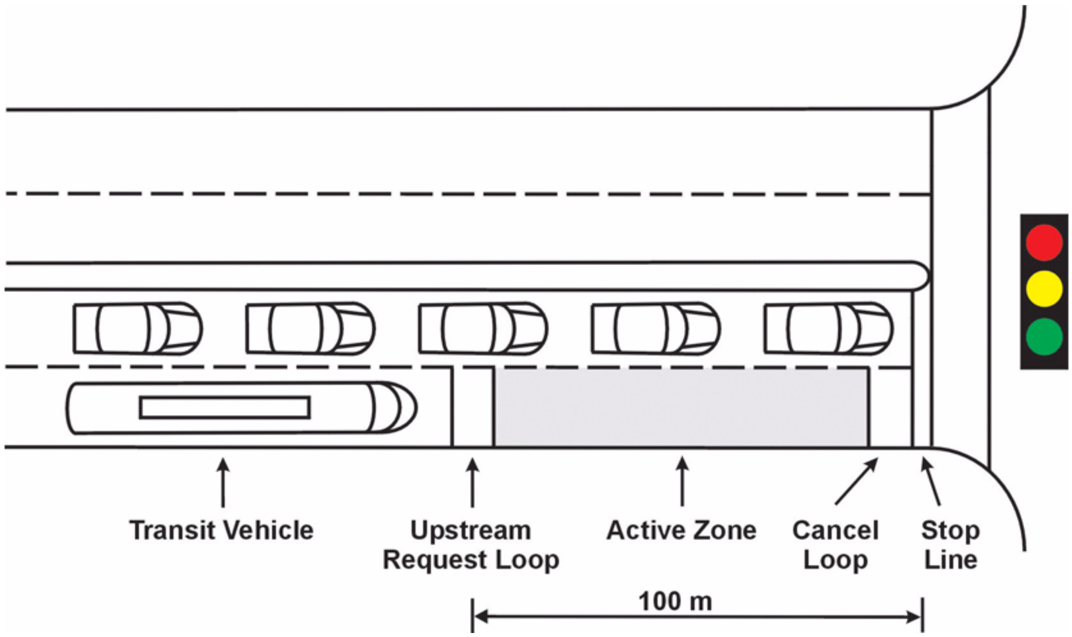


Figure 3.31 Request and cancel loops and active zones

For transit-corridor green extension, a decision point is defined. It may refer to the number of seconds before the end of the transit-corridor green (e.g. 12 seconds). Alternately, the decision point may be defined based on an interval number (e.g. react at the start of interval #3).

If either of the "zones is active" (i.e., for either transit route direction) at the time of the decision point for transit-corridor green extension, the green extension algorithm will begin with an initial fixed green time period for the transit corridor. This is followed by demand-dependant extensions (1 or 2 seconds depending on the controller type) for the transit-corridor green. The extensions are served consecutively until the zone is cleared (i.e., streetcar passes the cancel loop) or until a maximum number of extensions are provided.

An additional decision point is defined for the truncation. If the zone is active at the time of this decision point, the signal will also switch to local control. The signal timing will be altered to shorten the cross-street green time, and hasten the provision of green to the transit corridor. The amount of green time that will be subtracted from the cross street is a set value defined per intersection, ranging from 2 to 6 seconds after minimum walking time in the study area.

Those decision points for transit-corridor green extension and cross-street green truncation are defined for each intersection after pedestrian walking time and signal priority can be provided in successive cycles if the "zone" is still active.

Other Canadian Developments and Comparison with International Experience

There have been other TSP deployments in Canada, including:

- Isolated deployments of intersection control activated by approaching buses typically for buses entering an arterial from off-street terminals or subdivision secondary streets (e.g. Eglinton Bus Station, Edmonton);
- Recent corridor deployments, involving a limited number of TSP-equipped intersections in Quebec City, Longueuil, and Calgary;
- Recent or ongoing deployments of new sophisticated municipal traffic control systems that include TSP in Peterborough and York Region, both partially funded through Transport Canada ITS deployment grants;
- Deployment of TSP for the TransLink's Richmond and Granville Street Rapid Bus lines that involve the deployment of Transit ITS components in the form of real-time customer information system and TSP;

Most deployments of TSP in Canadian cities have been of the “unconditional” type, where priority is granted to any transit vehicle once detected upstream of the intersection. Although many TSP deployments in the U.S., Europe and Japan are also of the “unconditional” type, there have been recent implementations of “conditional” TSP and “adaptive” TSP. For example, conditional TSP has been implemented at 150 intersections in Portland, Oregon, where priority is granted based on the schedule adherence of the approaching bus (i.e. give priority if bus is late). In Japan, the UTMS 21, a “next generation” adaptive traffic management system, includes PTPS (Public Transport Priority System) as one of its subsystems. The PTPS/UTMS21 system has been applied to a number of Japanese cities including Tokyo, Nagoya, Hamamatsu, and Sapporo.

Another international development is the ongoing development of the NTCIP Signal Control Priority Standard in the U.S. that will cover Emergency Vehicles, Transit, and Light Rail. The NTCIP Standard contents include: concept of operations, functional requirements, dialogs and sequences (interface specifications), data dictionary, and test procedure.

3.5 Traffic responsive operation

3.5.1 Traffic responsive control

Traffic responsive signal control has the capability to select the most appropriate pre-defined traffic signal timing plan in response to the prevailing traffic condition. A typical traffic responsive control operation maintains a library that consists of a number of different traffic signal timing plans. The signal timing plans in the library define cycle times, phase splits, phase sequences, and offset times. These library plans are generally developed using off-line traffic signal optimization software and then stored electronically in the central computer and/ or field controller as part of a timing plan library for selection based on traffic parameters measured by vehicle detectors.

Historical traffic volume data collected at different times of the day and different days of the week may be used to develop signal timing plans for normal traffic conditions. For certain abnormal traffic conditions such as traffic incidents, adverse weather conditions, or unexpected traffic congestion, arbitrary traffic volumes may be generated as input data for the optimization software to define traffic signal timing plans. Traffic responsive plans may be created to address periodic traffic issues related to sporting events, theatres releases, shopping centres, etc. Traffic responsive signal control may also trigger an early execution or extension of set traffic signal timing plans.

The traffic parameters are sampled from tactical points throughout the controlled area network, and then used in the selection of timing plans under traffic responsive control. These parameters include volume, occupancy, queue length, or a combination of these three factors. Some strategies under this categorization define plan selection rules to decide the most suitable traffic signal plan solely based on these detector parameters. Other traffic responsive control strategies define specific traffic conditions, rather than detector parameters in memory, and identify the current traffic condition based on the traffic data using pattern matching techniques or simple mathematical forecasting methods. Switching of traffic signal timing plans are typically allowed when minimum sustained threshold conditions are met.

3.5.2 Traffic adaptive control

Traffic adaptive signal control is the most sophisticated and proven technique for traffic signal control at present. This type of signal control includes computational processes to define the optimal traffic signal timing that usually minimizes average vehicle delay time. For this computation, a typical traffic adaptive signal control strategy requires a traffic prediction model, which has the capability to proactively estimate the near future traffic conditions using traffic data collected at specific link locations. For traffic signal timing optimization, techniques such as hill-climbing, dynamic programming and genetic algorithms have been used for adaptive signal control strategies.

One type of traffic prediction model projects traffic arrival patterns at the stopline based on the measured traffic arrival patterns on the link upstream. Since traffic flow exiting one link approach doesn't progress uniformly as one platoon through the link downstream, a platoon dispersion model may be applied to simulate the behavioural traffic flow depending on the link approach length and the modeling method being used. These traffic prediction models simplify or ignore the detailed behaviour of vehicles, such as lane changing, passing, and acceleration/braking for computational efficiency.

Real-time traffic signal control schemes

Real-time traffic signal control is one of the advanced features that make traffic adaptive control strategies distinct from ordinary pre-timed control. Based on the scheme adopted, the framework of traffic adaptive signal control may take different forms.

A “cyclic” real-time control scheme adjusts traffic signal timing parameters based on the traditional cyclic framework. This type of traffic signal control can modify phase splits in response to real-time traffic demand changes while maintaining a constrained cycle time. However, depending on the magnitude of traffic changes, some cyclic adaptive control strategies allow adjustments of cycle time and offset by up to only a few seconds in a single cycle time (Hunt 1981 and Bretherton 1998).

An “acyclic” real-time control scheme doesn't explicitly consider the traditional cyclic signal control concept, but decides only the optimal phase switching times during the pre-determined optimization period (Gartner 1983 and Henry 1983). The term “optimization period” is different from cycle time in that one signal phase may be provided more than once or not provided at all in one period.

One clear advantage of the acyclic control scheme over the cyclic method is the flexibility in adjusting traffic signal timing to respond to large and rapid changes of traffic demands. On the other hand, cyclic control scheme has relative benefit over the acyclic scheme in maintaining coordination between adjacent traffic signals.

More recent acyclic traffic adaptive strategies have accommodated the multi-level traffic signal control concept to improve their traffic signal coordination efficiency. The suggested multi-level concept first optimizes traffic signal timing plans at the entire network level and uses the outcome to set constraints when subsequently optimizing the local intersection traffic signal timing (Donati 1984, Gartner 1995, and Head 1992)

Advantages of adaptive and real-time signal control include:

- Signals are continually determined for current conditions
- Eliminates the need to update the library of timing plans
- Ability to adjust to unusual traffic conditions (incidents, holidays, etc.)
- Timing plans do not need to be continually monitored and changed

Disadvantages include:

- High cost associated with the need for extensive surveillance (i.e. detectors need to be installed on all major links in the system)
- Loss of control over timing plans calculated and implemented by the system (ITE 2007)

3.5.3 Analysis under traffic responsive operation

Flow and saturation flow considerations are largely independent of the type of signal control. The evaluation procedures described in Section 4.6 "Evaluation" on page 4-95, however, apply fully only to fixed-time signal control. The application of these methods to traffic responsive operation requires a knowledge of the variability of cycle times and green intervals. Since many types of traffic responsive control exist, only general advice, applicable to traditional fully- or semi-actuated operations, can be given. The equipment and software used for these two control techniques, however, also vary.

If traffic on all approaches is light or moderate, then actuated signal operation should almost eliminate the overflow delay. This assumes that the setting of unit extensions and other operational parameters is correct. The degrees of saturation for the critical lanes should be nearly equal to the flow ratios:

$$\Sigma x_j = \Sigma y_j = Y$$

where: x_j = degree of saturation for the critical lane in phase j, with

$x_j = q_j / C_j$ (Section 4.7.2 on page 4-97)

y_j = flow ratio for the critical lane in phase j, with $y_j = q_{jadj} / S_j$ ("Lane flow ratio" on page 4-85)

Y = intersection flow ratio ("Intersection flow ratio" on page 4-85), and

q_j = arrival flow in the critical lane of phase j (pcu/h) (Section 3.1.1 on page 3-14)

C_j = capacity of the critical lane of phase j (pcu/h) (Section 4.7.1 on page 4-96)

S_j = saturation flow in the critical lane of phase j (pcu/h) ("Critical lanes" on page 3-60).

Table 3.27 "Determination of the average cycle time for traffic actuated operations," on page 3-79 indicates that for low degrees of saturation the average cycle time is merely the sum of the minimum green intervals and intergreen periods for all phases. The Table also shows that when the arrival flows on all approaches are near or at congestion levels, the operation of the traditional gap seeking semi- or fully actuated controllers will approximate a fixed-time control mode. Consequently, the maximum green intervals and the maximum cycle time can be used, and the operation analyzed as if it were in a fixed-time control mode. For the situations in between, the average cycle time is approximated by a linear relationship between the minimum and maximum cycles. Theoretically, the condition that the average number of arrivals is just accommodated in each phase can be used to derive the average cycle time but the actual average cycle time is normally longer.

Average green intervals are then allocated in proportion to the flow ratios for critical lanes.

If the arrival flow on only one approach is congested, the green interval for that phase will frequently reach its maximum setting. The cycle time equation for medium flows may still be used as an approximation. The longest possible green interval for the congested critical lane is then applied and the remaining available green time allocated to the other phases in proportion to the flow ratios for their critical lanes.

Table 3.27 Determination of the average cycle time for traffic actuated operations

Condition	Cycle time
if $Y \leq 0.6$	$c = c_{min} = \Sigma_j (g_{min} + I)_j$
if $0.6 < Y < 0.95$	$c = 2.71\, c_{min} - 1.71\, c_{max} - 2.86\, (c_{max} - c_{min})\, Y$
if $Y \geq 0.95$	$c = c_{max} = \Sigma_j (g_{max} + I)_j$

1. **Source:** Adapted from Akcelik 1994
2. **Note:** These relationships may not be applicable to other traffic responsive control techniques.
3. **Where:** Y = intersection flow ratio
   c = cycle time (s)
   g_{min} = minimum setting of the green interval (s)
   I = intergreen period (s)
   j = phase counter
   c_{min} = minimum cycle time (s), determined as the sum of minimum green intervals and intergreens for all phases
   c_{max} = maximum cycle time (s), determined as the sum of maximum green intervals and intergreens for all phases.

The best advice with respect to traffic responsive signals, especially if their functions differ from the traditional traffic actuated or semi-actuated operation, is to determine the cycle time and the duration of individual phases by direct observation whenever possible. It is especially important where pedestrian or left-turn actuations determine the immediate cycle times. If frequent, these events will result in an underestimation of delays, queues and other features determined on the basis of the cycle times from *Table 3.27*.

CHAPTER

4

PLANNING AND DESIGN

4.1 Introduction

Planning and design based on appropriate technical parameters are crucial steps in providing a transportation network that is safe and effective. This applies to new intersections and to intersection improvements. This Guide addresses all aspects of planning and design, excluding detailed geometric design. Key technical elements in planning and design are listed as follows, together with sources for this information:

- Principal considerations and the essential input variables for the planning or design tasks are described in Chapter 3: "Analysis" on page 3-13.
- The composition of individual phases and the cycle structure discussed in "Minimum green interval" on page 3-58 are not known at the beginning of the analytical process and they must be initially estimated.
- Arrival flows for the applicable design period must be available, either from direct surveys or derived from transportation demand models.
- Saturation flows must also be at hand, determined from local surveys (described in Chapter 5) or, for design, calculated from the regional values of the basic saturation flow using the adjustment procedures of Sections 3.2.3 on page 3-26 to 3.2.14 on page 3-54. For planning purposes, saturation flow estimates and recommendations discussed in Section 3.2.13 on page 3-52 may be applied.

What is the Most Accurate Way to Plan an Intersection?

A signalized intersection is a complex system in its own right, and it is also typically part of a network of signalized intersections. When the 2nd Edition of the Guide was published in 1995, there were only very limited software tools available for network analysis. That situation has changed completely - a range of packages is available, and many jurisdictions have chosen to focus heavily on the operational network perspective these packages provide.

However, along with the increased capabilities of these software packages comes significantly increased complexity in coding requirements. There are numerous parameters in Synchro and similar packages that are time and location-specific. With the focus on progression and the network perspective, the user's focus shifts away from the individual intersection. Nonetheless, the individual intersection is the fundamental building block of the network. It is essential that each intersection function efficiently and safely, independent of other signalized locations in the vicinity, before any network constraints are imposed on its operation, other than perhaps the cycle length that is utilized. The CGSI is a methodology that enables the user to accurately plan each intersection, by first calibrating to existing conditions, using a transparent process in which the effects of parameter changes are readily apparent.

With the InterCalc software, the user can plan the network in Synchro to obtain good network flow, and then export the data to InterCalc for enhancement of specific intersections, in order to obtain accuracy in intersection design and network operation.

4.2 Signal timing design

The main part of the design task involves the determination of the cycle time, the phase structure, and the allocation of the time available for traffic movements within that cycle to individual phases. Chapter 6: "The Process: Examples" on page 6-143 **includes three complete examples of the calculation process: one involving a very simple intersection configuration and two representing realistic intersection set-ups composed of a variety of lane types.**

The procedures require lane-by-lane arrival flows ("Traffic Flow" on page 3-13) and lane-by-lane saturation flows ("Saturation Flow" on page 3-23) for individual phases.

The tentative composition of individual phases as well as the structure of the whole cycle, including related intergreen periods and lost times ("Timing Considerations" on page 3-55), are needed as direct input. Pedestrian flows, minimum walk intervals ("Pedestrians" on page 3-67) and clearance periods are also necessary. If applicable, special requirements for transit vehicles and bicycles ("Bicycles" on page 3-70 and "Transit vehicles" on page 3-71) must be known.

4.2.1 Allocation of arrival flows to phases

In addition to the allocation of intersection movements to individual lanes ("Flow allocation to lanes" on page 3-18), the movements and lanes must be allocated to phases. Drivers approaching an intersection must select a lane that legally allows the movement in the desired direction. If they still have a choice, they also judge the degree of difficulty or advantages associated with individual lanes. The assignment of lanes and phases to movements attempts to represent that choice but, because of the complexity of these decisions, it can only be approximated. The process is often iterative.

Movements during one phase

It is advisable to analyze the departure pattern even for situations where only one phase is available for a given movement. In addition to the green interval, left-turning vehicles may discharge during the intergreen period, and the right-turning vehicles during the intergreen period or on the red interval (RTOR). Straight-through movements do not usually receive any additional discharge benefits.

Those portions of arrival flows that discharge during periods of the phase other than the green interval must be estimated. This almost always involves left turns on intergreen ("Left turns on intergreen period (LTOI)" on page 3-20). Right turns on the red interval ("Right turns on red interval (RTOR)" on page 3-21) are frequently ignored in the initial design and treated as a capacity bonus in the evaluation stage ("Capacity of approach lanes for vehicular traffic" on page 4-96). Right turns on intergreen are considered only where they conflict with pedestrian traffic, and only where pedestrian flows are greater than approximately 3000 ped/h. Local conditions must be considered since in areas with higher pedestrian volumes, the right turn capacity is significantly constrained during all green indications. This may result in vehicles using the intergreen period to turn right or vehicles may not be able to turn as a result of pedestrians crossing late in a phase (including the intergreen).The effect is usually negligible for lower pedestrian flows.

The initial estimates of arrival flows that discharge during periods other than the green interval should be kept low. These flows are then excluded from the initial calculation of the cycle time and green intervals by subtracting them from the lane arrival flows to be applied for the determination of the lane flow ratio and the green interval. The additional non-green intervals flows are, however, added to the lane capacity during the evaluation stage.

Departure flows from shared lanes must be analyzed with movements that can legally discharge during periods other than green intervals, as well as movements that are allowed only during green intervals. In most instances of a shared right-turn and through lane, for example, it is assumed in the initial calculation that the full arrival flow discharges only during the green interval, and the RTOR are considered a capacity bonus.

Movements during more than one phase

If more than one phase is available, the arrival flows must also be split among the phases available for that movement. In most cases, no more than two phases are involved (Figure 4.1). Saturation flows for the same direction of travel may be significantly different in each of the phases (left-turns in example b, left- and right-turns in examples b and c).

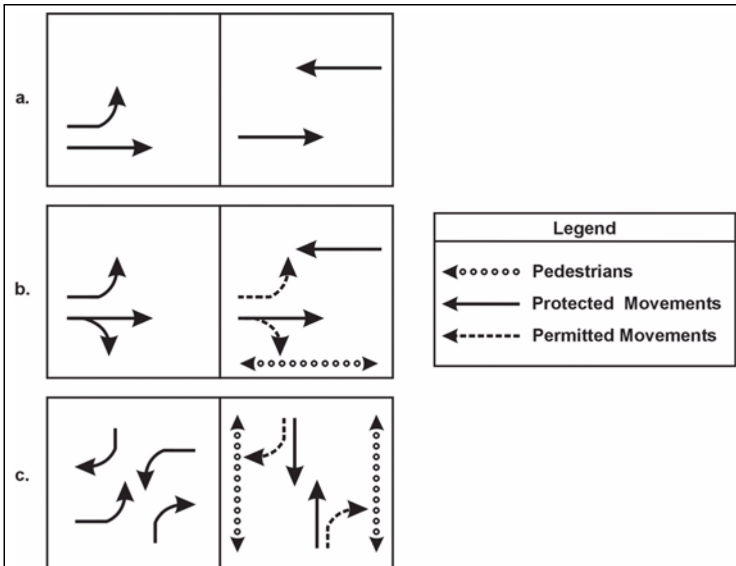


Figure 4.1 Examples of phase configurations and lanes that allow movements during two phases.

The following procedures may be applied:

a. *iterative:* depending on the type of phase, flows are divided in an arbitrary manner. For example, if a protected left-turn phase is a leading phase, as in Figure 4.1b, most, if not all of the left turns are assigned to this protected phase. The green interval for the following permissive left-turn phase is designed solely on the basis of the straight-through movements. The straight-through movement in Figure 4.1a may be split in an arbitrary fashion. If necessary, the flows are then adjusted in the next design iteration after the evaluation of capacity and other criteria.

b. *using flow ratios:* a decision regarding which of the movements or lanes will govern is required. For instance, in Figure 4.1a, the flow ratios for the left-turn lane in the first phase and the flow ratio for the straight-through movement westbound are calculated ("Lane flow ratio" on page 4-85). The flow ratio for the straight-through movement eastbound is calculated as if it took place during one phase.

One option is to make the first phase as long as necessary to accommodate the left turns. The flow ratio for the straight-through lane in that phase must therefore be equal to or less than the flow ratio for the left-turn lane. The remainder of the flow ratio for the straight-through lane is then assigned to the second phase. The lane with the highest flow ratio in the second phase, in this case either the westbound or eastbound straight-through lane, will then be the critical lane for that phase.

Another option involves making the straight-through lane westbound the critical lane. For the second phase, the flow ratio of the westbound lane is equated with the flow ratio for the straight-through lane eastbound. The difference between the flow ratios of the total westbound movements and its portion that takes place during the second phase is then assigned to the first phase. The flow ratio of the left-turn lane is compared to the flow ratio of the straight-through lane and the lane with the higher flow ratio becomes the critical lane in the first phase.

4.2.2 Flow ratio

Lane flow ratio

The flow ratio for a given lane is determined as:

$y_i = q_{iadj} / S_i$

where:

y_i = flow ratio for lane i

q_{iadj} = adjusted arrival flow in lane i (pcu/h) = (q_i - RTOR - LTOI - RTOI)

S_i = adjusted saturation flow of lane i (pcu/h).

The flow ratio reflects the proportion of the cycle time required to discharge a given arrival flow at a given saturation flow rate.

Intersection flow ratio

Under the assumption that all lane arrival flows depart during the green interval at full saturation flow, the lane with the highest flow ratio determines the duration of the green interval for that phase. This lane is identified as the *critical lane* for the phase ("Critical lanes" on page 3-60). Each phase may have only one critical lane.

The intersection flow ratio is then the sum of the critical lane flow ratios for all phases:

$Y = \Sigma_j y_{ij} = \Sigma_j (q_{ij} / S_{ij})$

where:

Y = intersection flow ratio

y_{ij} = flow ratio for the critical lane i in phase j

q_{ijadj} = adjusted arrival flow of critical lane i in phase j (pcu/h)

S_{ij} = saturation flow of critical lane i in phase j (pcu/h)

Σ_j= summation over critical lanes in phases j (one critical lane for each phase).

If $Y \geq 1.0$, the considered input variables result in exceeding the capacity of the intersection as described by the input variables used. In that case, the cycle structure, the composition of individual phases, the allocation of flows to lanes and movements to phases must be examined.

Numerous variations of the worksheet or spreadsheet used to calculate the intersection flow ratio exist. The InterCalc software computes this variable. The following form (Table 4.1) includes the common features.

Table 4.1 Example of a calculation format for lane and intersection flow ratios

Lane number	Direction	Phase number	Adjusted lane arrival flow q	Adjusted lane saturation flow S	Lane flow ratio $y = q/S$	Flow ratios for critical lanes y_{crit}
						$Y = \Sigma\, y_{crit}$

A numerical example is illustrated in Chapter 6: “The Process: Examples” on page 6-143.

4.2.3 Cycle time

Minimum cycle time for vehicular flows

The minimum cycle time calculation assumes that the intersection will be served at exactly capacity conditions (Webster and Cobbe 1966):

$$c_{min} = L / (1 - Y)$$

where:

c_{min} = minimum cycle time (s)

L = intersection lost time (s) (“Intersection lost time” on page 3-65)

Y = intersection flow ratio (“Intersection flow ratio” on page 4-85).

Optimum cycle time for vehicular flows

The optimum cycle time should accommodate the arrival flows at a better quality of service than the available capacity and minimize the overall intersection delay. The formula below gives approximate results that should be tested during the evaluation stage by determining the true value of intersection delay (Stewart 1992). Where more than two phases are involved or vehicular traffic approaches congested conditions, this test is especially important. The formula is based on a fixed relationship to the minimum cycle time, as follows (Webster and Cobbe 1966):

$$c_{opt} = (1.5\, L + 5) / (1 - Y)$$

where:

c_{opt} = optimum cycle time (s)

L = intersection lost time (s) (“Intersection lost time” on page 3-65)

Y = intersection flow ratio (“Intersection flow ratio” on page 4-85).

Since the form of the resulting function around this optimum cycle time is relatively flat, the cycle times between 0.75 c_{opt} to 1.5 c_{opt} are generally considered practical. The relationship between the minimum and optimum cycle times with respect to capacity and delay is illustrated in Figure 4.2.

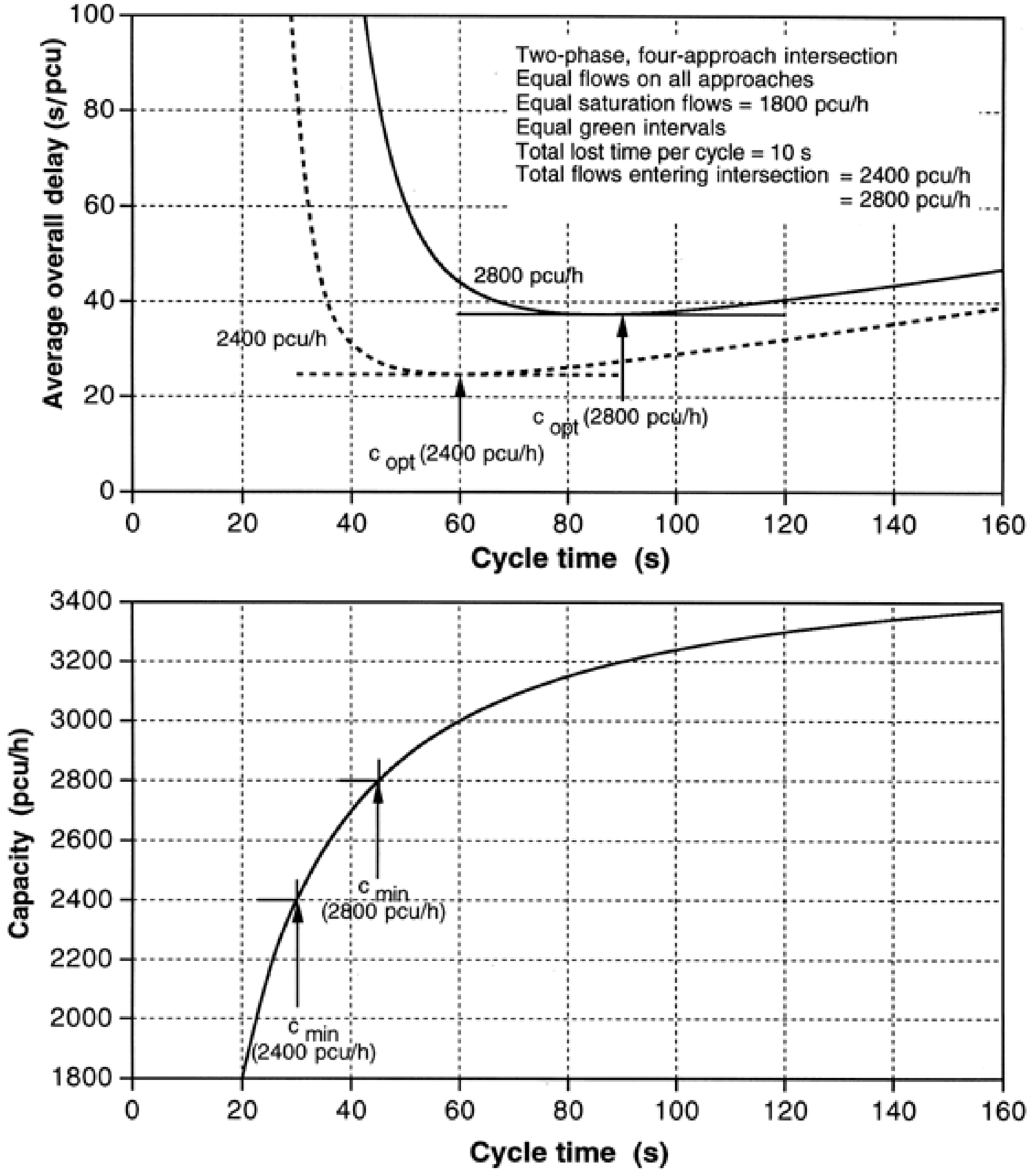


Figure 4.2 Relationship among cycle time, capacity and delay.
Source: based on Webster and Cobbe 1966

Maximum cycle time

There are practical limits to the duration of a cycle. Long cycles involve long red intervals for some movements and, as a consequence, long delays for drivers and pedestrians. A cycle time of 90 s is typical, and a value of 120 s is usually considered a practical upper limit, with 140 s used under exceptional conditions by some jurisdictions.

Minimum cycle time for pedestrians

The vehicular cycle time must not be shorter than the cycle time required for pedestrians:

$$c_{ped\ min} = \Sigma_j \max (w_{min\ i} + w_{clear\ i})_j$$

where:

$c_{ped\ min}$ = minimum cycle time required for pedestrians (s)

$w_{min\ i}$ = minimum pedestrian walk interval for crosswalk i (s) (Section 3.4.1 on page 3-67)

$w_{clear\ i}$ = pedestrian clearance period for crosswalk i (s) ("Pedestrian clearance period" on page 3-69)

$\max (w_{min\ i} + w_{clear\ i})_j$ = maximum of the sum of the minimum walk interval plus the corresponding pedestrian clearance period in each phase j (s) ("Pedestrian phase requirements" on page 4-89).

Cycle time selection

Normally, all of the above cycle times are calculated. The selected cycle time must not violate the lower bounds required for pedestrians and vehicles, and should not exceed a locally acceptable upper limit. Moreover, it should also reflect design objectives. In many instances, it must also conform to the system cycle time in the adjacent parts of the network in order to support effective progression. After the determination of the applicable evaluation criteria, the cycle time may be reconsidered and the calculation repeated.

4.2.4 Green allocation

Depending on the objectives of the design, many allocation criteria can be used, such as:

- balancing flow ratios;
- minimizing degrees of saturation for major directions;
- minimizing total intersection overall delay;
- minimizing average overall delay for some movements only;
- minimizing total person delay;
- providing delay penalty for "shortcutting" vehicular traffic;
- balancing probabilities of discharge overload;
- minimizing delays to pedestrians; or
- various forms of queue management.

The process is iterative in most instances and starts with the balancing of the flow ratios for individual phases. The value of the selected criterion is then determined for each lane and phase, and the green intervals adjusted. If necessary, the process is repeated. Only the flow-ratio-balancing method is discussed here in detail.

Defining green intervals by balancing flow ratios

This procedure uses flow ratios for the critical lanes (Section 3.3.2 "Green interval" on page 3-58 and Section 4.2.2 "Flow ratio" on page 4-85). First, the total time available in the cycle for the allocation to green intervals is determined as:

$$\Sigma g_j = c - \Sigma I_j$$

where:

Σg_j = total green time available in the cycle (s)

c = selected cycle time (s)

I_j = intergreen period following phase j (s).

This total available green time is allocated in proportion to the flow ratio of the critical lanes for the corresponding phases and the intersection flow ratio:

$$g_j = \Sigma g_j \, y_j / Y$$

where:

g_j = green interval for phase j (s)

y_j = flow ratio for the critical lane in phase j (Section 3.3.2 "Green interval" on page 3-58 and Section 4.2.2 "Flow ratio" on page 4-85)

Σg_j = total green time available in the cycle (s)

Y = intersection flow ratio (Section 4.2.2 "Flow ratio" on page 4-85).

Pedestrian phase requirements

The total duration of a phase plus the following intergreen period must not be shorter than the longest of its walk interval plus the corresponding pedestrian clearance period:

$$(g_j + I_j) \geq \max\,(w_{min\ i} + w_{clear\ i})_j$$

where:

g_j = green interval for phase j (s)

I_j = intergreen period following phase j (s)

$w_{min\ i}$ = walk interval for crosswalk i (s)

$w_{clear\ i}$ = pedestrian clearance period for crosswalk i (s)

$\max\,(w_{min\ i} + w_{clear\ i})j$ = maximum of the sum of the walk interval and pedestrian clearance period (s).

4.3 Planning applications

The main objective of the planning task is to identify the required intersection features which would allow reasonable freedom for the signal timing design in the future. Furthermore, it should not preclude the use of cycle structures and phase compositions that may reasonably be anticipated.

In most instances, the cycle time need not be calculated. Table 4.2 provides cycle time estimates that would be adjusted only if a simplified evaluation process indicates that they are not suitable.

Table 4.2 may also be applied for a preliminary cycle time estimate where required in the determination of saturation flow adjustments (3.2.4 on page 3-31).

Table 4.2 Cycle time estimates for planning applications[1]

Intersection type and cycle structure	Arrival flows and intersection flow ratios at intersection approaches		
	light $Y \leq 0.6$	medium $0.6 < Y \leq 0.85$	heavy $Y > 0.85$
simple	60	80	90
elaborate	80	100	120
complex	90	110	120

1. **Notes:** Y = intersection flow ratio (Section 4.2.2 "Flow ratio" on page 4-85)
"simple" = two phases at three- or four-approach intersections
"elaborate" = three phases at four-approach intersections
"complex" = three or more phases at intersections with unusual geometry.

4.4 Pedestrians, bicycles and transit vehicles

4.4.1 Pedestrians

Pedestrian timing requirements are discussed in “Minimum cycle time for pedestrians” on page 4-88 and “Pedestrian phase requirements” on page 4-89.

4.4.2 Bicycles

Bicycles are considered to be vehicles in all Canadian jurisdictions. The vehicular timing requirements described in “Timing Considerations” on page 3-55 and “Signal timing design” on page 4-83 therefore apply. Where cyclists are required to dismount when using a pedestrian crosswalk, they are considered to be pedestrians and “Pedestrian clearance period” on page 3-69, “Minimum cycle time for pedestrians” on page 4-88 and “Pedestrian phase requirements” on page 4-89 should be used.

4.4.3 Transit vehicles

Person flows and vehicle occupancy are described in Section 3.1.3 on page 3-16; the influence of streetcar tracks in the pavement surface on vehicular saturation flow in “Pavement conditions” on page 3-27; the impact of transit stops on saturation flow in “Transit stops” on page 3-37; and the total overall person delay as an evaluation criterion in “Non-vehicular delay” on page 4-105.

Where special phases or other priority measures for buses, trolley buses, streetcars or Light Rail Transit (LRT) vehicles are included, their minimum timing requirements may influence the duration of individual phases and the cycle time design (Yagar 1993, Brilon and Laubert 1994).

The time for special transit phases may be added to the cycle time. In systems that must maintain a fixed cycle time, a special transit phase may be inserted into the cycle in response to the presence of a bus or a streetcar. In that case, the time needed for the transit interval is provided by shortening or omitting one of the regular vehicular phases in those cycles in which a special transit phase is actuated. The minimum requirements of the cycle time, minimum green intervals, minimum walk intervals, as well as the necessary intergreen periods and pedestrian clearance periods must not be violated. The effect of this special function on the operation of the affected movements should be analyzed in detail.

The saturation flow expressed in buses (bus/h) indicates an average saturation headway of 4.16 s (3600/865 See “Special lanes” on page 3-52). Where long transit vehicles, such as articulated buses or streetcars use the intersection, there are different strategies associated with the transit signal intervals. The European practice is to not necessarily provide a transit signal interval that would include the whole period during which the total length of the vehicle travels through the intersection. Once the front part of the vehicle has crossed the potential conflict area, the remaining body of the vehicle acts as a barrier and the transit signal indication can be terminated. Other jurisdictions require a transit signal interval that allows transit vehicles to clear the intersection fully. Rapid transit modelling should be discussed with the relevant staff in jurisdictions which have unusual transit operations such as streetcars, etc.

4.5 Traffic responsive operations

Traditional traffic-actuated operation employs a set of pre-programmed rules that determine the sequence and duration of signal intervals based on gap threshold values. Computer technology, however, allows more advanced traffic responsive control system functions that can take into account a number of objective functions and constraints. For instance, transit-actuated phases (Section 3.4.4 on page 3-71) **may be used with special decision logic that reflects the immediate state of traffic and of the control system.**

The procedure for design of the basic parameters for traffic responsive control methods have, so far, been conceptually identical to those for fixed time operation.

Lost time is largely independent of the type of operation, except for complex cycle structures with skip-phase features. The cycle time in most traffic-actuated operations, however, varies from cycle to cycle. In the calculations, it is represented by the average cycle time. This average cycle time is not known and must be approximated as described in Table 3.27 on page 3-79. It is, however, still advisable to calculate the optimum cycle time (“Optimum cycle time for vehicular flows” on page 4-86) since it provides an insight regarding potential capacity problems.

The maximum cycle time used will depend largely on the control method but should not exceed 120 s, with 140 s to 160 s in justifiable instances. If the calculated optimum cycle time is greater, it is likely that a *traffic actuated* signal operation will approximate a fixed time control mode. Typical cycle times used for different circumstances are listed in Table 4.3. These values, however, may not be applicable to all types of traffic responsive or traffic adaptive control.

Table 4.3 Design cycle times for traffic actuated operations[1]

Cycle structure	Maximum cycle time setting (s)
simple	80 (90)
elaborate	100 (120)
complex	120 (140 or 160)

1. **Notes:** “simple” = two phases; “elaborate” = three or four phases; “complex“= multiphase operation.
The numbers in () denote values for exceptional situations.

The minimum green intervals should comply with the values listed in Table 3.23 on page 3-58. Where pedestrian signal indications are needed, the minimum duration of a phase is also determined by the minimum walk and pedestrian clearance requirements as described in “Pedestrians” on page 3-67 and “Pedestrian phase requirements” on page 4-89. This condition may apply only to the phases and cycles for which a pedestrian push button device has been activated.

4.5.1 Signal coordination and other system considerations

The objective to minimize delays or the number of stops by the progressive movement of traffic through adjacent intersections requires some form of signal coordination (Teply and Hunt 1988). A common cycle time is normally needed but is not necessary in some adaptive network control systems. The most critical intersection in the system usually governs and its cycle time is applied to all other intersections. Where the choice is relatively unconstrained, somewhat longer cycle times may be favoured since they allow for an easier design of progressive movement.

Signal coordination introduces additional conditions regarding phase composition and cycle structure. Leading or lagging protected left-turn phases, although they may not be needed from the point of view of a single intersection operation, may be beneficial since they allow different offsets for each direction of a two-way roadway. Dwell times at transit stops and the location of bus or streetcar stops may also influence the cycle time, green interval or offset design (Yagar 1993, Brilon and Laubert 1994, Jacques and Yagar 1994). Upstream signal operations influence downstream intersections and, in some instances, the build-up of queues at downstream intersections may lead to blocking of upstream locations. “Gridlock” represents extreme conditions of this kind.

As a result of signal coordination and other system considerations, the initial timing design for an individual intersection in a larger system may have to be reiterated. Many computer network programs calculate a nearly optimal fixed-time signal setting for a given cycle time and for a given set of cycle structures in an urban arterial network under an objective function that combines delays and stops. Some programs also take into account public transit operations in the network or the reassignment of traffic flows that takes place as a result of signal controls. Other programs allow an integration of freeway and arterial operations. Some on-line control hardware/software systems continuously adapt the timings to the measured and projected traffic conditions in the network. Control strategies are further discussed below.

The objectives of local intersection analysis, planning or design should be examined regarding their network implications and subordinated to broader systems goals.

System considerations are important but exceed the scope of this Guide.

4.5.2 Traffic responsive control

Traffic responsive signal control has capability to select the most appropriate pre-defined traffic signal timing plan in response to the prevailing traffic condition. Properly programmed responsive control can provide significantly better traffic signal control over pre-timed operations with good response to traffic condition changes in signal network areas that can be subject to significant unpredictable traffic variations.

However, this type of signal control needs to be periodically updated, so that the plan selection rules and the library of timing plans may better account for visible alterations in the possible longer term trend in traffic patterns, and/or road network and land use changes. Stale rules and plans can cause traffic responsive signal control strategies to select and provide inadequate signal timing plans which will compromise the efficiency of the signal operation.

Another potential downside of this type of signal control is in handling of traffic signal timing plan transitions. Since this type of signal control continuously seeks the best traffic signal timing plan, which may have different cycle and offset times from the regular plan. This operation can result in inefficient spent time for transitioning from one signal timing plan to another. Most traffic responsive control operations restrict the frequency of traffic signal timing changes and consequently sacrifice the flexibility of a more responsive operation.

4.5.3 Traffic adaptive control

Traffic adaptive signal control strategies continuously monitor traffic conditions and generate optimized traffic signal timings in response to traffic demand changes. The efficiency of traffic adaptive control has been proved by a number of previous studies. In effect, a very robust 24/7, 365 days a year, traffic signal operation is provided. Reported benefits include reduced traffic delay, stops, fuel consumption, and emissions (Moore 1999 and Luk 1982). Also, traffic adaptive control was found to increase safety by reducing the number of rear-end collisions opportunities through improved signal coordination (Hicks 2000).

Since the development of earlier adaptive strategies (Hunt 1981 and Sims 1979), the latest adaptive strategies have incorporated more sophisticated forms of traffic prediction models and signal timing plan optimization techniques mainly due to the recent technology improvements (Gartner 1995 and Head 1992). The current trend in adaptive signal control research and development enables more realistic modeling of traffic flow and optimization of traffic signal timing. However, the reliance on the vehicle detector data becomes very important in a successful operation. Deficiencies in the traffic detection system may degrade the performance of the adaptive control operation significantly. Thus, efforts must be made in maintaining the detection system to guarantee the optimal operation of traffic adaptive signal control. Furthermore, traffic adaptive signal control strategies may require intensive efforts to maintain their optimal parameter settings.

Additionally, traffic adaptive signal control strategies typically require larger initial capital investments. New facilities to implement a typical adaptive system include advanced type signal controllers, communication links/interconnections, and a reliable vehicle detection system. Compared to traffic responsive control, adaptive systems may require more system detectors to capture a more comprehensive set of data.

4.6 Evaluation

The evaluation process involves criteria that describe how the intersection operates. The selection of the evaluation criteria is governed by the given problem and by the objectives of the analysis, planning or design tasks.

4.6.1 Evaluation criteria

The evaluation criteria described in this Section of the Guide represent the most frequently used measures of effectiveness of signal operation. They are listed below with an * denoting those criteria for which the survey procedures are included in Chapter 5: “Surveys” on page 5-123:

a. Related to capacity

- capacity
- degree of saturation
- probability of discharge overload and overload factor*

b. Related to queueing

- average overall delay in s/pcu*
- average overall delay in s/veh
- average stopped delay*
- average stopped delay and Level of Service
- total overall person delay
- average delay to pedestrians
- number of stops
- queue at the end of red interval
- average queue reach*
- maximum probable queue reach*
- queue at the end of evaluation period (oversaturated conditions)
- maximum queue reach during congestion period

c. Other operational and environmental criteria

- fuel consumption
- cost
- emissions

4.7 Evaluation criteria related to capacity

These evaluation criteria represent the cornerstones of capacity analysis. They have a great deal of descriptive power by themselves and are needed as input for the determination of more complex evaluation measures.

4.7.1 Capacity of approach lanes for vehicular traffic

Capacity of lane i during phase j is determined as

$$C_{ij} = S_{ij}\, g_{e\,j} / c$$

where:

C_{ij} = capacity of lane i in phase j (pcu/h)

S_{ij} = saturation flow of lane i in phase j (pcu/h)

$g_{e\,j}$ = effective green interval of phase j (s) (“Green interval and effective green interval” on page 3-58)

c= cycle time (s).

For lanes that discharge vehicular traffic during more than one phase, the total capacity is the sum of the capacities of that lane during all individual phases (“Movements during more than one phase” on page 4-84). The departures that take place during periods other than green intervals must also be added (“Special flow considerations” on page 3-19). Therefore, the general formula for capacity is:

$$C_{ij} = \Sigma_j\, S_{ij}\, g_{e\,j} / c + n\, X_a$$

where:

C_{ij} = capacity of lane i in phase j (pcu/h)

Σ_j = summation over phases j

S_{ij} = saturation flow of lane i in phase j (pcu/h)

$g_{e\,j}$ = effective green interval of phase j (s) (“Green interval and effective green interval” on page 3-58)

c= cycle time (s)

n = number of cycles per hour = 3600 / c

X_a = average number of passenger car units that can depart from lane i during periods other than green intervals in one cycle, such as discharge during red intervals or intergreen periods (3.1.7 on page 3-19). These flows may have been discounted from the total arrival flow during the timing design stage (“Movements during one phase” on page 4-83) but must be added to lane capacity.

Crosswalk capacity

Under forced flow conditions, the unidirectional pedestrian flow can be as high as 4000 to 5000 pedestrians per hour per metre of the effectively used width of the walkway (ped/h/m) (Navin and Wheeler 1969, Fruin 1971, Brilon et al 1993). Such crowded conditions are usually tolerated only at special events and evacuations. The recommended maximum *unidirectional* pedestrian flow for the design of typical crosswalks in Canadian urban centres is 2000 ped/h/m, which corresponds to a density of about 0.5 person/m^2 at a free speed for most pedestrians. This value, however, must be reduced where an unfavourable mix of the two opposing directions of the pedestrian flow exist. To maintain a similar quality of pedestrian flow as for the unidirectional conditions, the resulting value may be only 0.75 of the recommended unidirectional flow. (Navin and Wheeler 1969, TRB 1994). Local investigations are advisable.

Very high pedestrian arrival flows on sidewalks leading to the crosswalk may result in the crowding of sidewalk corners. These situations where the sidewalk corner is a constraint to the crosswalk flow may be analyzed using the procedure described in Chapter 18 of the Highway Capacity Manual (TRB 2000).

4.7.2 Degree of saturation

This ratio indicates the proportion of available lane capacity that is used under the given cycle structure. It is determined as the ratio of the arrival flow and the maximum departure flow or capacity:

$$x_i = q_i/C_i$$

where:

x_i = degree of saturation of lane i

q_i = arrival flow of lane i (pcu/h), also referred to as the hourly volume (V)

C_i = capacity of lane i (pcu/h).

The degree of saturation is also known as V/C ratio, or volume-to-capacity ratio.

4.7.3 Probability of discharge overload and overload factor

Probability of discharge overload

This criterion recognizes the fact that even in those cycles during which fewer vehicles than the cycle capacity arrive, there still may be discharge overload because of a residual queue from the previous cycle. It can be approximated as the probability of dependent events in two consecutive cycles, derived from the Poisson distribution (Teply 1993a) as:

$$P_{\text{discharge overload}} = 1 - [P(X \le X_c)]^2$$

where:

$P(X \le X_c)$ = arrival overload probability, i.e., the probability that the number of arrivals in a cycle (X) will be equal or less than cycle capacity X_c, and

$$P(X \le X_c) = \Sigma\, m^i\, [e^{(-m)}] / i!$$

where:

$P(X \leq X_c)$ = probability of the number of arrivals in a cycle (X) being less than or equal to cycle capacity (X_c), with

$X_c = C/n$ = cycle capacity (pcu/cycle)

C = capacity (pcu/h)

n = number of cycles in an hour = 3600 / c

c = cycle time (s)

Σ_i = summation for i = 0,1,2,3,..., X_c

m = average number of arrivals distribution per cycle time, calculated as:

m = q / n = q c / 3600 (pcu/cycle), with

q = arrival flow (pcu/h).

Cycles measured in Edmonton confirmed that the practical value of the effective green interval may be conservatively assumed to be by one second longer than the displayed green interval (Sonnenberg 1995). The distribution of cycle capacities followed a normal distribution with a constant standard deviation of approximately one passenger car regardless of the absolute average capacity value.

The values of probability of discharge overload can also be determined from Figure 4.3. As illustrated, the probability of discharge overload greater than about 0.65 practically represents a continuous overload. The reason for it not being 1.0 when the average arrival flow exceeds capacity is that this evaluation starts with the assumption of no overload at the beginning. Consequently, there is some probability that the first several cycles will have no overload. The higher the oversaturation, and the longer it lasts, the greater is the likelihood of a continuous overflow.

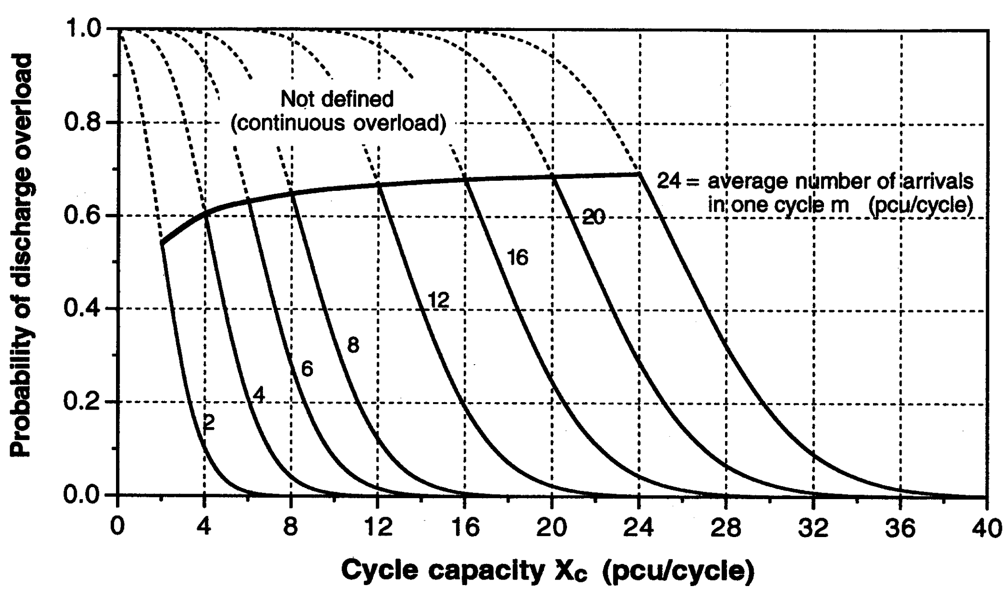


Figure 4.3 Probability of discharge overload.

Overload factor

For existing intersections operating under the conditions evaluated, the probability of the discharge overload can also be approximated by a surveyed overload factor (Teply 1993a). The survey method is described in Chapter 5: “Surveys” on page 5-123. It is defined as the number of cycles in which a residual queue was observed, divided by the total number of observed cycles:

$$OF \cong P_{\text{discharge overload}}$$

where:

OF = overload factor = n_o / n with

n_o = number of overloaded cycles, that is the cycles in which the green interval was fully used at the saturation level and a queue of at least one vehicle formed at the stop line at the end of amber

n = total number of consecutively surveyed cycles.

4.7.4 Level of Service

This section describes a methodology for assigning letter grades to qualitatively assess the operation of an intersection. To assist in clarifying the arithmetic analysis associated with intersection operations, it is often useful to refer to “Level of Service”. The term Level of Service implies a qualitative measure of traffic flow at an intersection. It is dependent on the vehicle throughput of an intersection. This type of simple summary can be extremely useful in communicating the results of intersection analyses to lay audiences and others.

Several guides including the Highway Capacity Manual as well as various software packages based on these guides, have defined measures to assign letter grades. In past editions, the Guide made reference to assigning letter grades to qualitatively measure the intersection operational conditions based on the old HCM average stopped delay. It is noted that the HCM 2000 now bases Level of Service on control delay. However, it is believed that there is more logic in basing the Level of Service primarily on the volume to capacity ratio (degree of saturation).

Level of service calculation

The overall volume-to-capacity ratio for an intersection is calculated as the sum of the critical lane flow ratio for each phase multiplied by the ratio of cycle length to total effective green time.

$$V/C_{overall} = \Sigma\frac{q_\phi}{S_\phi} \times \frac{c}{g_e}$$

where:

$\Sigma\frac{q_\phi}{S_\phi}$ = the sum of the critical lane flow ratios for each phase

Level of service designation

Table 4.4 describes the ranges of V/C ratio that define each level, and the characteristics of each level.

The table also lists the current HCM thresholds for Level of Service (based on control delay). A delay-based Level of Service can produce very conservative results. A small number of vehicles turning from a minor street may have a disproportionate impact on Level of Service because of the high delay. There needs to be consideration in the method for the range of typical operating conditions at signalized intersections.

The nomenclature in Table 4.4 originated from the recommendations in the 1965 Highway Capacity Manual. The delay per vehicle threshold tends to address driver discomfort and fuel consumption for the time lost at an intersection due to deceleration, stopping and acceleration. Delay is considered to be a subjective entity that can vary for different individuals, situations and locations. A relationship between the driver discomfort and delay may not be linear. It should also be noted that the delay is also a time specific parameter. A delay acceptable under today's increasing congestion may not have been acceptable in the past when there were fewer vehicles on the road. A similar trend is expected in the future. In comparison, a ratio of discharging volume to capacity, in most conditions, represent the traffic utilization of the available roadway capacity. Compared to delay, a V/C value is discreet and definitive, and gives a clearer picture of the amount of available capacity remaining in an intersection independent of the time, user, location etc.

This Guide, basing Level of Service on V/C, presents an accurate representation of the operation of the intersection. The V/C is a fixed quantity that speaks to a logical assessment of the relationship between traffic volumes and approach capacity.

Table 4.4 Levels of Service for Signalized Intersections[1]

Level of Service	Features	V/C Ratio[2]	HCM Control Delay (s/pcu)
A	Almost no signal phase is fully utilized by traffic. Very seldom does any vehicle wait longer than one signal cycle. The approach appears open, turning movements are easily made and drivers have virtually complete freedom of operation.	0-0.59	≤ 10
B	An occasional signal cycle is fully utilized and several phases approach full use. Many drivers begin to feel somewhat restricted within platoons of vehicles approaching the intersection.	0.60-0.69	> 10 and ≤ 20
C	The operation is stable though with more frequent fully utilized signal phases. Drivers feel more restricted and occasionally may have to wait more than one signal cycle, and queues may develop behind turning vehicles. This level is normally employed in urban intersection design.	0.70-0.79	> 20 and ≤ 35
D	The motorist experiences increasing restriction and instability of flow. There are substantial delays to approaching vehicles during short peaks within the peak period, but there are enough cycles with lower demand to permit occasional clearance of developing queues and prevent excessive backups.	0.80-0.89	> 35 and ≤ 55
E	Capacity is reached. There are long queues of vehicles waiting upstream of the intersection and delays to vehicles may extend to several signal cycles.	0.90-0.99	> 55 and ≤ 80
F	Saturation occurs, with vehicle demand exceeding the available capacity.	1.0 or greater	> 80

1. **Source**: TRB 2000
2. Recommended for use in the Guide

In summary, the overall level of service of an intersection depends on both the V/C and delays. Long delays can occur when V/C ratios are acceptable, if:

- The cycle length is long;
- A specific lane group has a long red phase; and/or
- The signal progression for the movement is poor.

On the other hand, high V/C ratios (>1.00) can accompany short delays if:

- The cycle length is short;
- The specific lane group has a short red phase; and/or
- The signal progression for the movement is good.

See Section 4.8.1 "Vehicle delay" on page 4-101.

Thus a saturated condition does not necessarily imply a long delay, and vice versa. It is important to assess both the v/c ratio and delay to fully evaluate the operation of a signalized intersection. A well-designed intersection should have acceptable volume to capacity ratios and delays for all movements.

4.8 Criteria related to queueing

These measures of effectiveness are a direct result of the interrupted nature of the vehicular traffic flow caused by the operation of signal control. Some vehicles in the arrival flow are stopped by the red signal indication and form a queue. They are delayed until the green signal indication appears and the preceding vehicles in the queue ahead move. When more vehicles arrive than the number that can be discharged during the green interval, vehicles that were unable to depart must wait for the next or more subsequent cycles.

4.8.1 Vehicle delay

It is customary to express vehicular delays as averages in s/pcu or s/veh for individual intersection lanes or movements. Although it is also possible to determine the average delay per person, it is more useful to calculate the total person delay for individual intersection movements. The total person delay reflects not only the waiting time but also how many persons have been waiting and therefore considers the types of vehicles and their occupancies. Pedestrian delays may serve as an evaluation or design criterion in some instances.

Average overall delay in s/pcu

Average overall lane delay in s/pcu

Since delay is related to the driver's perception of "waiting at a signal", its various forms are frequently used as the major characteristics of the quality of signal operation. The literature on the subject is very rich (Webster and Cobbe 1966, Robertson 1969, Huber and Gerlough 1975, Ackcelik Rouphail 1994 and a great number of other published sources).

All of the following equations apply to a single lane. Lane changing within the intersection approach space may somewhat equalize delays in the lanes that carry the same intersection movements.

The basic equation for estimating the average overall delay is as follows:

$$d_o = k_f\, d_1 + d_2$$

where:

d_o = average overall delay (s/pcu)

k_f = adjustment factor for the effect of the quality of progression, with

$k_f = (1 - q_{gr}/q)\, f_p \,/\, (1-g_e/c)$ (calculated values are also given in Table 4.5), and

q_{gr} / q = proportion of vehicles arriving during the green interval, with

q_{gr} = average number of arrivals during green interval

m = average number of arrivals per cycle

f_p = supplemental adjustment factor for platoon arrival time from Table 4.5.

d_1 = average overall uniform delay (s/pcu)

d_2 = average overflow delay (s/pcu), with

$d_1 = c\,(1 - g_e/c)^2 \,/\, [2\,(1 - x_1\, g_e/c)]$ and

$$d_2 = 15 t_e\,[(x-1) + \sqrt{(x-1)^2 + 240x/(C t_e)}]$$

where:

c = cycle time (s)

g_e = effective green interval (s) ("Green interval and effective green interval" on page 3-58)

x_1 = degree of saturation (maximum value of 1.0)

x = degree of saturation ("Degree of saturation" on page 4-97)

C = capacity (pcu/h) ("Capacity of approach lanes for vehicular traffic" on page 4-96)

t_e = evaluation time (min) ("Analysis period, evaluation time, design period, period of congestion, and transit assessment time" on page 3-17)

The overflow delay component reflects the influence of both random and continuous overflows (Kua 1990). It usually need not be considered for those lanes in which:

a. S > 400 pcu/h and x < 0.6, or

b. S > 1000 pcu/h and x < 0.8.

Table 4.5 Progression adjustment factor k_f [1]

g_e/c	Arrival type					
	AT1	AT2	AT3	AT4	AT5	AT6
0.2	1.2	1.0	1.0	1.0	0.8	0.8
0.3	1.3	1.1	1.0	1.0	0.7	0.6
0.4	1.4	1.1	1.0	0.9	0.6	0.3
0.5	1.7	1.2	1.0	0.8	0.3	0
0.6	2.0	1.4	1.0	0.6	0	0
0.7	2.6	1.7	1.0	0.3	0	0
f_p	1.0	0.9	1.0	1.2	1.0	1.0

1. **Notes:** Arrival types are defined as follows:
   AT1: very poor progression; almost all vehicles arrive during the red interval
   AT2: poor progression
   AT3: random arrivals or platoons proportionately split between red and green intervals
   AT4: favourable progression
   AT5: good progression; most vehicles arrive during the green interval
   AT6: exceptionally good progression; for example, on one-way streets.
   The supplemental adjustment factor for platoon arrival time $\mathbf{f_p}$ reflects the effect of the time when the front of the main platoon arrives. It does not have any effect when the signal progression is either very good or very poor or when the vehicles arrive at random.

Source: adjusted from TRB 2000

The adjustment factor for the quality of progression k_f for isolated intersections is 1.0. In conditions for which the effect of signal coordination is known or can be reasonably estimated, for instance, from time to time, space diagrams (Teply and Hunt 1988, Teply and Evans 1989), its values must be calculated or taken from Table 4.5.

Figure 4.4 on page 4-104 illustrates the influences of input variables on the average overall delay. The formula gives a good representation of delay magnitudes but it is less reliable where the degree of saturation $x \geq 1.2$. The overall delay is relatively easy to measure with reasonable accuracy (Hurdle 1984, Teply and Evans 1989).

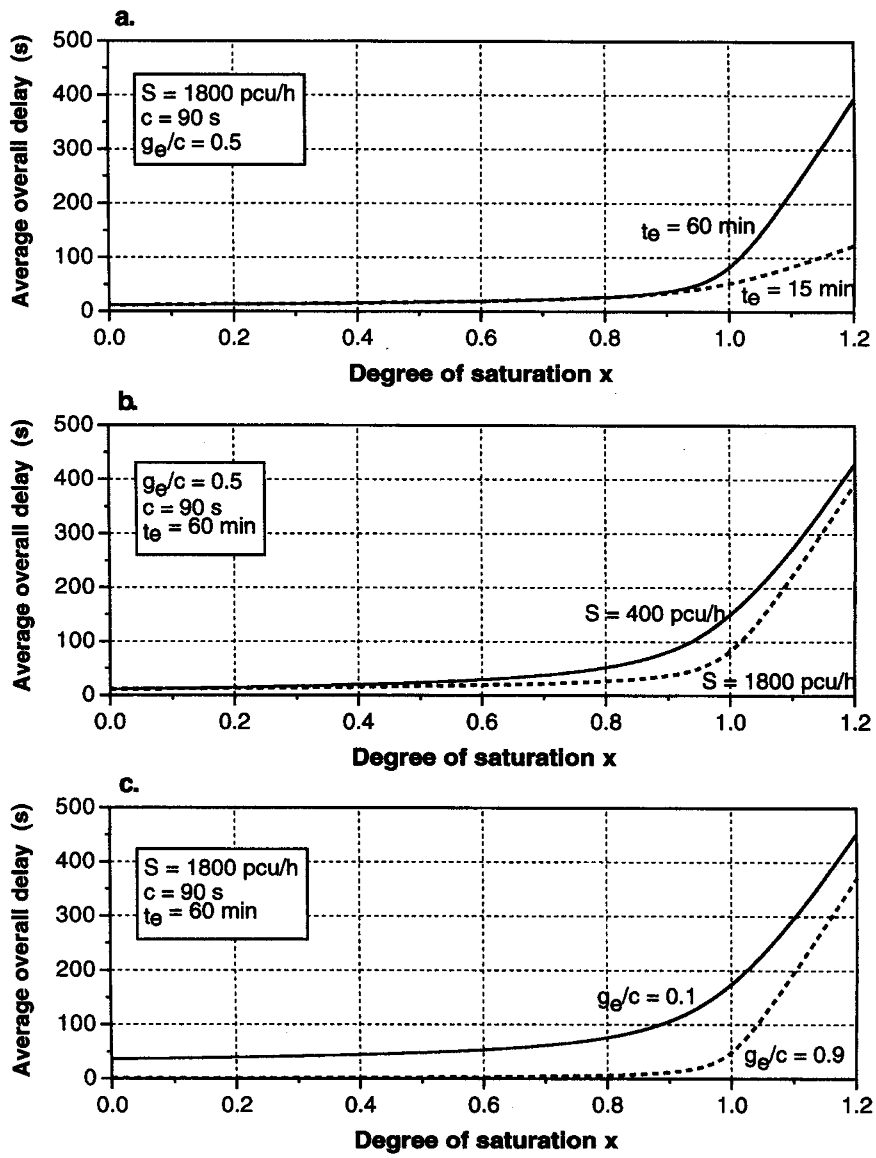


Figure 4.4 Influences on the average overall lane delay:
a. evaluation time t_e; b. saturation flow S; c. g_e/c ratio.

In complex overflow situations such as multiple peaking, the average overall delay may be determined from the queueing diagram (“Vehicular queues” on page 4-108) by discrete or graphical integration of the total delay. This is the area of the graphs in Figure 4.5 on page 4-108 and Figure 4.8 on page 4-112 between the arrival and departure flows divided by the total number of passenger car units departing.

Average overall intersection delay in s/pcu

The average overall intersection delay is calculated as the weighted average of the average overall lane delays for all lanes of the intersection:

$$d_{int} = \Sigma_j \Sigma_i q_{ij} d_{ij} / \Sigma_j \Sigma_i q_{ij}$$

where:

q_{ij} = arrival flow in lane i in phase j (pcu/h)

d_{ij} = average overall delay for vehicles in lane i in phase j (s/pcu)

$\Sigma_j \Sigma_i$= summation over individual lanes i and over phases j.

Average overall delay in s/veh

At locations where unusual traffic composition exists, it may be useful to determine average overall delay for vehicles rather than for passenger car units. The value of this delay type may be required for the determination of several other evaluation measures.

In situation where passenger vehicles constitute more than 85% of all vehicles, or where the non-linear overflow delay component is small relative to the value of the uniform delay, the conversion is not necessary because the difference is negligible for practical purposes.

Arrival flows expressed in veh/h are usually directly available from traffic counts. Saturation flows and capacity, however, are usually calculated and expressed in pcu/h, and their values in veh/h for the prevailing traffic composition must therefore be determined ("Saturation flow in pcu/h" on page 3-25). The average overall delay formula ("Average overall lane delay in s/pcu" on page 4-101) is then applied, with veh/h as the arrival flow and capacity units.

4.8.2 Non-vehicular delay

Total person delay

This measure allows the incorporation of public transport and different vehicle occupancies as a signal timing design criterion. This type of delay does not include pedestrian delay that must be calculated separately ("Average delay to pedestrians" on page 4-106).

The type of delay is expressed as *total* overall person delay, which is the sum of all person delays, not as an average overall person delay. The total person delay includes the deceleration time and is calculated as the delay to individual vehicle categories weighted by their average occupancy during the evaluation time. The transit peak period reflected by both the number of transit vehicles and their occupancies, however, rarely coincides exactly with the vehicular traffic peak. A period different from the evaluation time that represents how long the analyzed person flow lasts, may therefore be applied. The arrival *volumes*, that is the actual number of vehicles approaching the intersection during the *transit assessment time* ("Analysis period, evaluation time, design period, period of congestion, and transit assessment time" on page 3-17), not the arrival flows must be used, together with the average occupancies of individual vehicle categories during the assessment time.

The total overall person delay in a lane is calculated as follows:

$D_{person\ i} = d_{o\ veh\ i}\ \Sigma_k\ (V_{ki}\ O_{ki}) / 3600$

where:

$D_{person\ i}$ = total person delay in lane i (h)

$d_{o\ veh\ i}$ = average overall delay per vehicle ("Average overall delay in s/veh" on page 4-105) in lane i *based on transit assessment time* (s/veh) (Section 3.1.5 on page 3-17)

O_{ki} = average occupancy of vehicles of category k in lane i during the transit assessment time (person/veh)

V_{ki} = volume of vehicles of category k in lane i (veh/assessment time).

The sum of all these person delays in all lanes that receive the green signal indication in the given phase j is then the total person delay in that phase:

$D_{person\ phase\ j} = \Sigma_i\ D_{person\ lane\ ij}$

and, for the whole intersection, the sum of all person delays in all phases is:

$D_{person\ int} = \Sigma_j\ D_{person\ phase\ j}$

where:

$D_{person\ int}$ = total intersection person delay (h) during transit assessment time

i = lane counter

j = phase counter.

Average delay to pedestrians

Assuming random pedestrian arrivals at a given crosswalk and operation below the crosswalk or sidewalk capacity, the average pedestrian delay is independent of the pedestrian flows, and is calculated as:

$d_{ped} = (c - w)^2 / 2c$

where:

d_{ped} = average delay to pedestrians (s/ped)

c = cycle time (s)

w = walk interval (s).

The total pedestrian delay can be defined as:

$d_{ped\ int} = \Sigma\ d_{pedj} \times q_{pedj}$

where:

d_{pedj} = average pedestrian delay in crosswalk j (s/ped)

q_{pedj} = pedestrian volume in crosswalk j (ped/h)

4.8.3 Number of stops

The number of vehicles that are stopped at least once by the signal operation during the evaluation time can be derived with the assumption of a random arrival pattern as (Webster and Cobbe 1966):

$$N_s = k_f \, t_e \, q \, (c - g_e) / [60 \, c \, (1 - y)]$$

where:

N_s = number of passenger car units stopped at least once during the evaluation time (pcu). The resulting value must be capped at a maximum of:

$$N_s \leq q \, t_e / 60$$

representing the number of passenger car units arriving during the evaluation time.

k_f = adjustment factor for the effect of the quality of progression from Table 4.5 on page 4-103 and "Average overall lane delay in s/pcu" on page 4-101

q = arrival flow (pcu/h)

g_e = effective green interval (s)

c = cycle time (s)

y = lane flow ratio

$y = q / S$ capped at $y \leq 0.99$,

with

q = lane arrival flow (pcu/h)

S = saturation flow (pcu/h)

t_e = evaluation time (min)

This formula does not consider multiple stops. The derived number of stops therefore must not exceed the volume during the evaluation time. This condition applies similarly to any other time period used. For higher degrees of saturation characterized by a significant overflow delay component, some vehicles must stop more than once. For these instances, the more complex Australian equation is appropriate (Akcelik 1981).

Furthermore, in the k_f factor, the formula approximates the effect of signal coordination and progressive movement of vehicles through the intersection space. This factor is essential when the number of stops is subsequently used in the determination of fuel consumption and pollutant emissions. The resulting number of stops may range from zero for excellent progression to 2.6 times greater than the number calculated by the formula for random arrivals. This effect of the k_f factor, however, may result in the number of stops exceeding the volume during the evaluation time and must therefore be constrained as indicated.

4.8.4 Vehicular queues

Queueing criteria are usually applied to the lanes where queues may impede the operation of other lanes, such as left-turn bays, or for the lanes on approaches in which queues may block an upstream intersection.

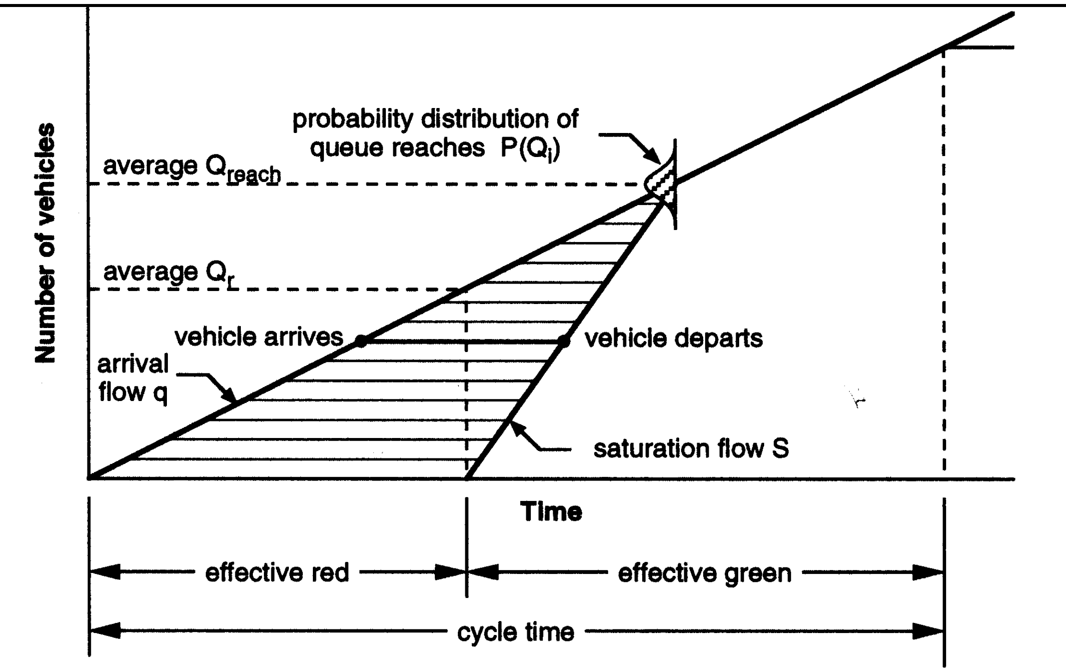


Figure 4.5 Queue types in undersaturated conditions represented in a queueing diagram. **Source:** Teply 1993b

The length of all queue types is normally expressed in passenger car units. If the length in metres is needed, the number of passenger car units is multiplied by the average space required for one passenger car unit, taken as 6.0 m. If the queueing space is critical and vehicular traffic flow composition includes a large number of longer vehicles, it is advisable to work with the observed average length of vehicles in individual categories. The queueing diagram in Figure 4.5 illustrates the types of queues included.

Queue at the end of the red interval

The average queue at the end of the red interval in undersaturated conditions with random arrivals is:

$$Q_r = q\,(c - g_e) / 3600$$

where:

Q_r = average queue at the end of the red interval (pcu)

q = arrival flow (pcu/h)

c = cycle time (s)

g_e = effective green interval (s).

Average queue reach

This type of queue is usually more important than the end-of-red queue because it identifies how far upstream a queue will stretch. It is measured from the stop line to the rear of the last vehicle joining the queue at the moment when the front departure shock wave catches up with the shock wave created by vehicles joining the queue. The reach of the queue relates directly to the available queueing space or distance. Since queues are subject to a high degree of randomness, exact determination is difficult. Two extreme cases are therefore calculated: the liberal and the conservative estimates (Teply 1993b).

A *liberal estimate* of the average queue reach in undersaturated conditions (Figure 4.5) can be determined as:

$$Q_{reach} = q\ (c - g_e) / [3600\ (1 - y)]$$

where:

Q_{reach} = estimate of average queue reach (pcu)

q = arrival flow (pcu/h)

c = cycle time (s)

g_e = effective green interval (s)

y = lane flow ratio = q / S, with

S = saturation flow (pcu/h).

A *conservative estimate* of the average queue reach in undersaturated conditions assumes that all vehicles arriving during a cycle must stop and join the queue, and is calculated as:

$$Q_{reach} = q\ c / 3600$$

where:

q = arrival flow (pcu/h)

c = cycle time (s).

With an increasing degree of saturation, the average queue reach calculated from the liberal formula approaches the value determined from the conservative estimate. Although good signal progression may substantially reduce queueing space requirements, this effect is normally not included in the calculations because it may constrain future design options.

Maximum probable queue reach

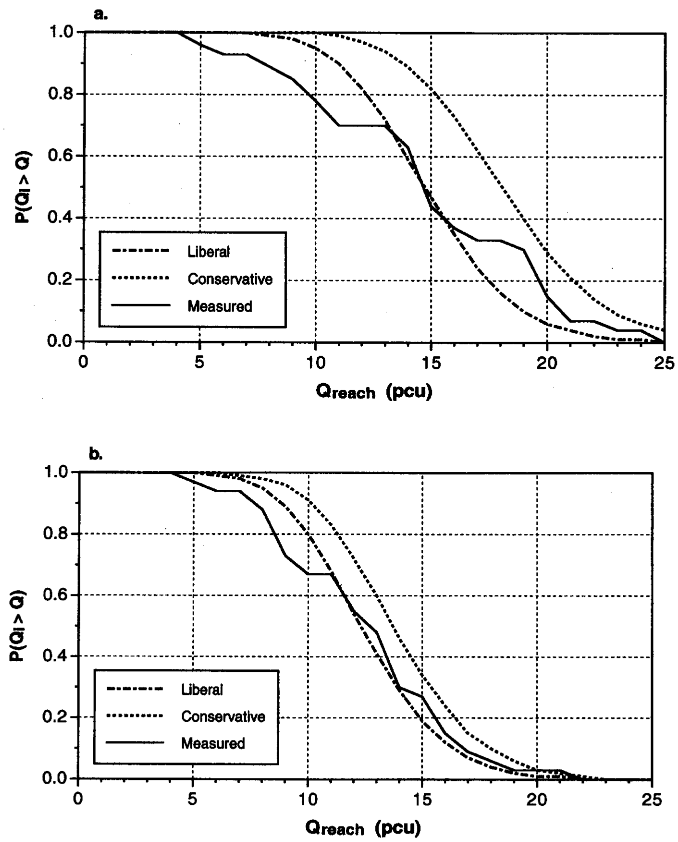


Figure 4.6 Typical examples of measured and calculated queue reach distributions: **a. for low degree of saturation (x = 0.75)b. for high degree of saturation (x = 0.90). Source:** Fung 1994

The maximum *probable* queue reach reflects the variability of arrivals from cycle to cycle. The concept is schematically illustrated in Figure 4.5 on page 4-108 by the small distribution pattern designated as $P(Q_i)$. The maximum probable queue reach in undersaturated conditions can be determined in a similar fashion as the probability of discharge overload ("Probability of discharge overload" on page 4-97). For design, the mean of the queue distribution should be taken as the conservative estimate of the average queue reach Q_{reach}. The actual maximum probable queue reach at the probability levels normally used usually falls between the two functions based on the liberal and conservative estimates (Figure 4.6).

The probability of a queue reach exceeding a given length (expressed as the number of passenger car units) may be approximated as:

$$P(Q_i > Q) = 1 - [P(Q_i \leq Q)]^2$$

where:

$P(Q_i > Q)$ = probability that a given queue reach Q will be exceeded, and

$P(Q_i \leq Q)$ = distribution of queue reaches with the calculated average queue reach Q_{reach} as the mean, and expressed in the cumulative form:

$$P(Q_i \leq Q) = \Sigma_j (Q_{reach})_j [e^{(-Qreach)}] / j!$$

with:

Q_{reach} = average queue reach estimate (pcu) from the conservative formula ("Average queue reach" on page 4-109)

j = summation parameter, representing queue states 0, 1, 2, 3,.... Q.

The graph in Figure 4.7 may be used to determine the maximum probable queue reach. The illustrated example shows, that for an average queue reach Q_{reach} = 16 pcu and a left-turn bay that is 120 m long and can therefore accommodate 120 m / 6m/pcu = 20 pcu, the probability of encountering a queue that reaches beyond the available space is 0.246, that is approximately 25%. For design purposes, the procedure can be reversed and the queue reach determined for the desired level of probability. For instance, in the example above, if the length of this left-turn bay is critical, its length can be determined from the probability that the queue will not stretch beyond the end of the bay in more than 5% of the cycles. For the average queue reach of 16 pcu and the probability level of 0.05, the required length of the left-turn bay is 24 pcu. Since each pcu needs 6.0 m, the length of the bay would be 6 m/pcu x 24 pcu = 144 m. The appropriate probability level is selected depending on the seriousness of the consequences of the queue extending beyond a critical point, and may be as low as 1% where an approach lane or the upstream intersection may be blocked. The probability level for conditions previously described would correspond to about 27 pcu x 6 m = 162 m.

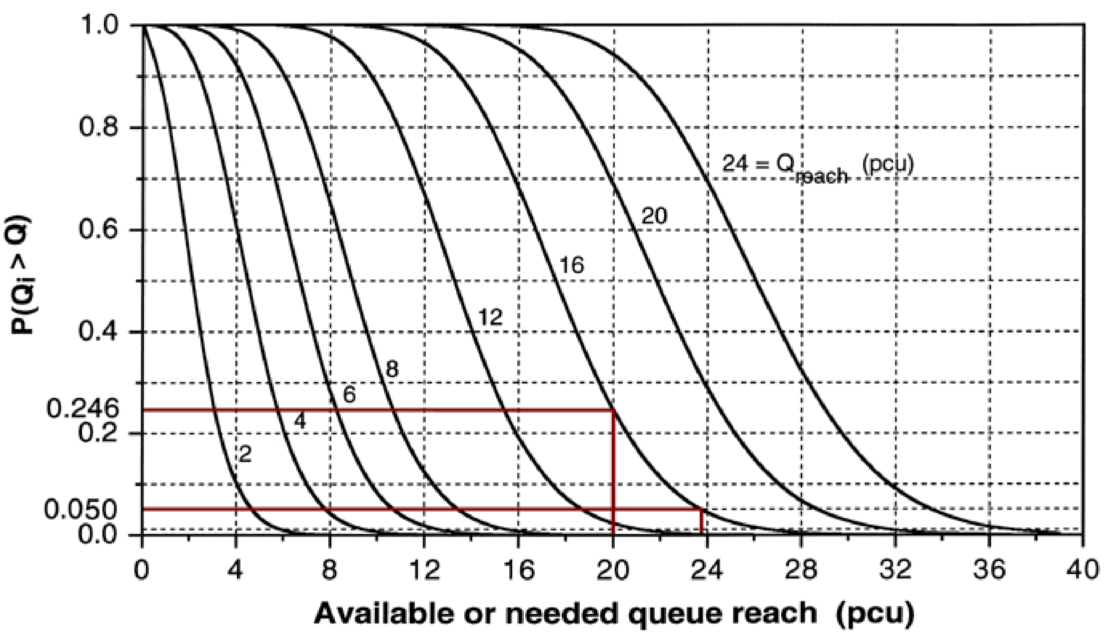


Figure 4.7 Maximum probable queue reach.**Source:** Teply 1993b

Queues in saturated conditions

The basic situation for oversaturated conditions is shown in Figure 4.8.

Queue at the end of evaluation period

The queue at the end of evaluation period in oversaturated conditions may be approximated as the sum of the continuous overflow queue at the end of the evaluation period and the queue at the end of the red interval:

$$Q_{te} = [t_e (q - C) / 60] + [q (c - g_e) / 3600]$$

where:

Q_{te} = queue at the end of the evaluation time (pcu)

t_e = evaluation time (min)

q = arrival flow (pcu/h)

C = capacity (pcu/h)

g_e = effective green interval (s).

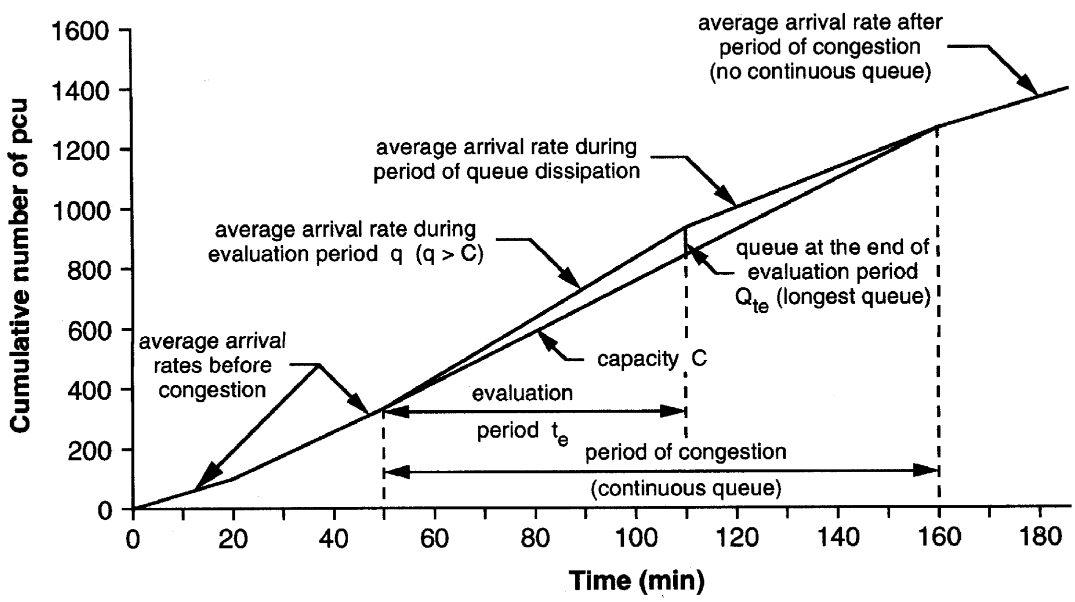


Figure 4.8 Queueing diagram for oversaturated conditions. **Source:** Teply 1991

Maximum queue reach during congestion period

The maximum reach of the queue during the evaluation or congestion period may be approximated as the sum of the continuous overflow queue at the end of the evaluation period ("Queue at the end of evaluation period" on page 4-111) and the conservative average queue reach estimate for undersaturated conditions.

The maximum queue reach during the congestion period is determined as:

$$Q_{max} = [t_e (q - C)] + Q_{reach}$$

where:

Q_{max} = maximum queue reach during the congestion period (pcu)

t_e = evaluation time (min)

q = arrival flow (pcu/h)

C = capacity (pcu/h)

Q_{reach} = conservative estimate of average queue reach (pcu) ("Average queue reach" on page 4-109).

Alternatively, the maximum probable queue reach for the desired probability level ("Maximum probable queue reach" on page 4-110) may be used instead of the average queue reach estimate.

4.9 Other operational and environmental criteria

Many other measures of effectiveness apply to the operation of signalized intersections. The analysis of a single intersection, however, is not well suited for the complexity of most of the criteria. Network models are more appropriate since they can account for the influence of the adjacent control measures and other system elements. The following measures should therefore be used with caution and only for comparisons of different signal timings, geometric design alternatives or for general planning applications.

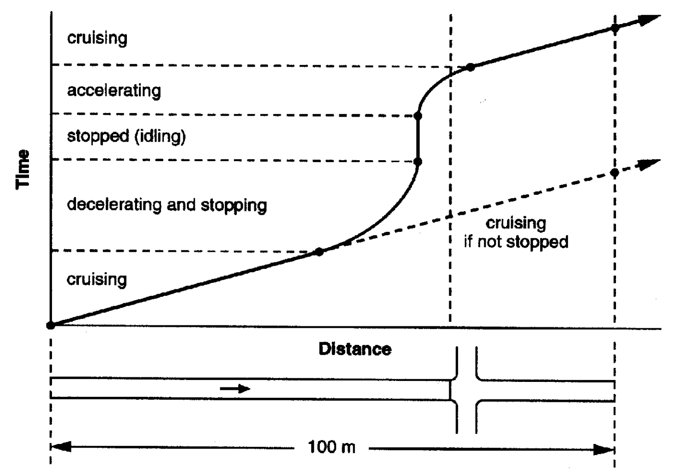


Figure 4.9 Stages of vehicle operation at a signalized intersection included in the fuel and pollutant emission model.

There is increasing interest in fuel consumption and emissions. Emissions include carbon monoxide (CO), carbon dioxide (CO_2), hydrocarbons (RHC) and nitrogen oxides (NO_x). A simplified time-space diagram which includes the deceleration, idling and acceleration process is shown in Figure 4.9.

Note: CITE is endeavouring to update the emissions analysis presented in the 2nd Edition. That update will be issued as a supplement to the CGSI.

4.10 Safety at Traffic Signals

The objective of this chapter is to provide a broad overview of the safety of traffic signals. This chapter is not a comprehensive review of road safety at signalized intersections and should not be used as such.

4.10.1 Overview

Canada, in its "Road Safety Vision 2010" document, has established a road safety vision that aspires for this country to have the safest roads in the world. This vision includes a specific objective of a 20% reduction in the number of road users killed or seriously injured in speed and intersection related crashes. In Canada, 27% of fatal collisions and 40% of serious injury collisions occur at intersections. Intersections on urban streets where the speed limit is 60 km/h or less and where traffic signals are prevalent are particularly dangerous. Forty-seven percent of all people killed and 57% of those seriously injured in intersection crashes incurred their injuries in urban locations. [Transport Canada, Road Safety Vision, 2002]

Traffic signals are traditionally known as devices that improve mobility by allocating right-of-way, and not as safety devices or collision countermeasures. That is not to say that traffic signals do not have safety impacts. In fact, the placement, design, operation and maintenance of traffic signals are very important considerations in the safe operation of intersections. To that end, the practitioner should have a basic understanding of road safety impacts as they relate to traffic signal installation and operations.

It should be noted that Traffic Control Signals are not always safer, such that they can increase rear-end collisions when installed when not warranted/justified.

4.10.2 Measuring Safety

"Safety" is a subjective term that is measured or expressed in terms of safety performance. The safety performance at a signalized intersection is a function of three measurable elements: 1) exposure (contact with hazards), 2) probability (the likelihood of a roadway hazard producing a collision), and 3) consequence (the resulting severity level if the hazard is encountered and causes a collision). These component parts of safety performance are further described as follows:

- Exposure: Measured by the volume of traffic (either motorized or vulnerable road users) entering the intersection or completing a particular movement.
- Probability: The likelihood of a hazard producing a collision is determined by the physical and operating characteristics of the intersection. For example, clearance intervals, head placement, angle of intersection, presence of turn lanes, signal coordination, etc.
- Consequence: Reflected in the severity of the collision (e.g., fatal, injury, or property damage only). The collision severity may also be predicted from the speed of travel, the type of collision (initial impact) and the elements located at the roadside.

Using safety performance as described above, the operational strategy can be measured and compared to alternatives. Ideally, the safety performance should be expressed as a quantitative measure, although at times this cannot be done and a qualitative measure of collision risk is acceptable.

The safety performance at a traffic signal should be expressed as the number of collisions (measured or expected) of a given severity for a particular entering volume and distribution (See Figure 4.10). This is different from the traditional "collision rate" which uses the sum of the entering volumes, and assumes that the collision rate is constant for all entering volumes.

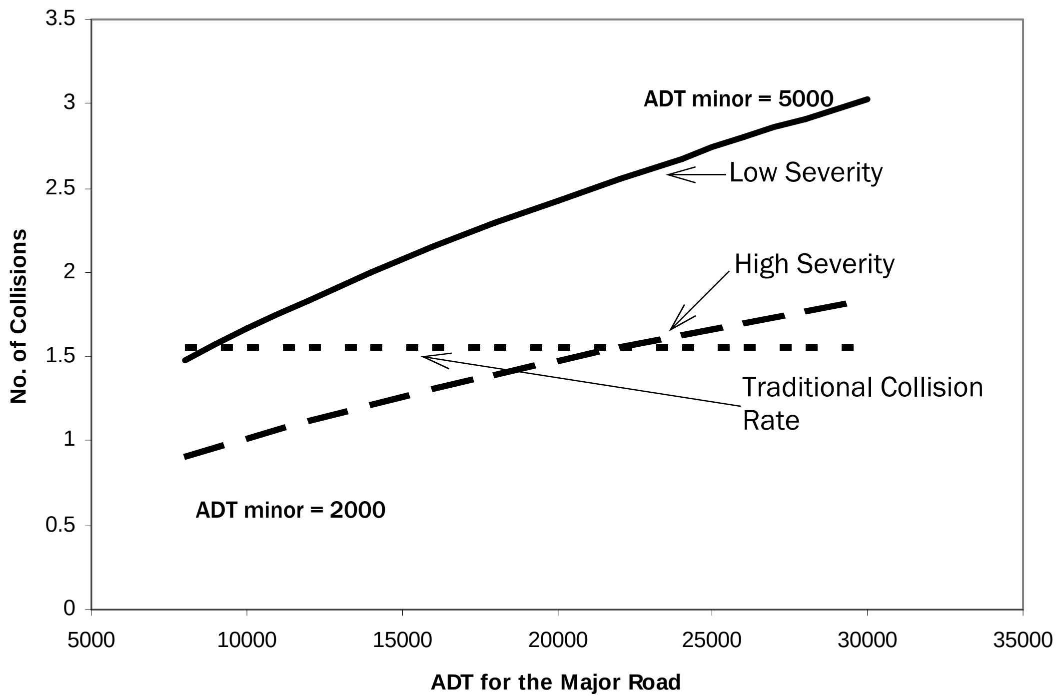


Figure 4.10 Number of collisions as a function of entering volume and distribution

For a more complete description of measuring safety and safety performance the reader is referred to Hauer (1997).

In those instances where the safety performance cannot be expressed in quantitative terms, then a qualitative scale may be used. This scale will typically use three categories of risk: high, medium and low. Assigning a road feature or condition to a category of collision risk is subjective and largely based on the knowledge and experience of the safety analyst. A useful taxonomy is provided in Table 4.7.

It should be noted that Figure 4.10 represents a generalized collision curve. Areas will have unique collision curves and local collision data should be used where available.

4.10.3 Explicit Consideration of Safety and Competing Objectives

The customary method of considering safety at traffic signal installations was to design and operate signals in compliance with all applicable standards and guidelines. This long-established method has evolved. Good practice today requires that practitioners explicitly consider the safety impacts of changes and alternatives in the design and operation of signals, and to understand that "safety" is a relative measure. While compliance with standards is desirable, it is only a starting point - explicit consideration of collision risk is also required.

The primary reason for explicitly considering safety at signalized locations is to allow practitioners to make more informed decisions regarding signalization and signal operations. At signalized locations there is inevitably a trade-off between the competing objectives of safety and mobility. Generally, modifications that are intended to enhance safety also negatively impact mobility. For example, introduction of a protected left-turn phase will increase safety, but also increase intersection delay. By explicitly considering safety through the collision risk, the practitioner can better decide what trade-offs are warranted.

Traffic signals may introduce new safety concerns while alleviating others, and the explicit consideration of safety also allows the practitioner to determine the net safety impacts. Signals may reduce collisions by separating conflicting movements temporally, accounting for sight distance limitations, and protecting crossing pedestrians. Signals also introduce collision risk by increasing rear end collision potential, increasing risk taking and driver frustration, while introducing fixed objects such as signal poles proximate to the traveled way. An understanding of these collision risks and a full accounting of them are essential elements in the explicit consideration of safety during traffic signal placement, design and operation.

The goal is to explicitly identify the impacts of signal installations or modifications, and to find a solution that balances safety and mobility impacts. In this process, it must be remembered that traffic signals are points of confluence for pedestrians, cyclists and motorists, and that the safety of all road users must be considered.

4.10.4 Safety During the Service Life of a Signal

Safety can and should be considered at all phases of signal planning, design, and operation and maintenance. Traditionally, signals have been installed based on satisfying a warrant that is based on traffic volumes entering the intersection, and some limited data on the physical characteristics of the intersection such as number of lanes, operating speed, urban or rural setting, etc. The safety impacts of signalization were not typically considered in the decision to install a signal, unless there was an unacceptable collision record that could not be improved except by signal installation.

Today, good practice dictates that the safety implications of signal design and installation be considered during all phases of the signalization process. New tools such as road safety audits and collision prediction models are available to practitioners. These permit explicit and meaningful analysis of safety throughout the life cycle of a traffic signal. Table 4.6 presents a framework for including safety in this life cycle process

Table 4.6 Framework for Including Safety in the Life Cycle of a Traffic Signal

	Stage of the Life Cycle			
	Planning (Decision to signalize)	Signal Design (including layout and timing)	Construction	In-service
Objective	Mitigation/Prevention	Prevention	Prevention	Mitigation
Methods of explicitly considering safety	Compare the predicted safety performance under signal control with those of the existing control, and alternative forms of intersection control	Road Safety Audits and Conformance checks	Road Safety Audits and Conformance checks	Compare safety performance of the signal to the expected safety performance of the signal via Network Screening
Tools	Collision prediction models, collision modification factors	Road safety audit guidelines, positive guidance reviews, design consistency checks	Road safety audit guidelines, positive guidance reviews, design consistency checks	Network screening tools, In-service road safety reviews, positive guidance reviews, traffic conflict studies

Planning

The planning stage of the signal life cycle is the most critical stage with respect to safety because the type of intersection control is a major predictor of safety performance. Consequently, the planning stage presents the greatest opportunity to incorporate safety into the intersection. The main consideration is whether the installation of a traffic signal is the best decision from a safety perspective. Alternative treatments, such as modern roundabouts, may provide the functional utility of a signal, with potential for improved safety.

The profession has moved away from the traditional “collision” warrant, and many jurisdictions now employ a comparative assessment of safety performance for different forms of intersection control using collision prediction models. The reader is referred to Stewart (2003) for a more detailed description.

Signal Design and Construction

As the design moves from functional through detailed design stages, safety should be explicitly considered. Basic design elements, such as the provision of turning lanes, the physical placement of signal heads to maximize visibility, or the use of break-away poles, are examples of how safety can be included in design.

The starting point for all signal design is compliance with standards, guidelines and recommended practices. The Transportation Association of Canada (TAC) Geometric Design Guide for Canadian Roads (TAC, 1999), and the TAC Manual of Uniform Traffic Control Devices for Canada (TAC, 1998) are primary references in this regard. Both of these documents promote uniformity and consistency in the design and application of various measures - a significant safety consideration.

The staging of construction and achieving the transition of the intersection from an unsignalized to a signalized state is another point in the life cycle at which safety should be taken into account. This stage presents particular concerns as drivers have the potential to be confused by visual cues present in the form of “dark” signal heads, installed but not yet operating, which can conflict with existing traffic controls.

During this phase of the life cycle, there is no collision record associated with the traffic signal since it has not yet been placed in service. During this phase, where collision prevention is the goal, the principle tools available to the practitioner are the Canadian Road Safety Audit Guide (TAC, 2001) and the positive guidance review (See 1.6 "Intersection Geometric and Control Elements" on page 1-6 for a description).

In Service

The safety of a signalized location, once it is placed in service, is accomplished through preventative maintenance programs and intersection monitoring. Regular maintenance includes such activities as lens cleaning, signal re-lamping, and conflict monitor checks. Good practice respecting preventative maintenance and the frequency of inspection are local policy issues that should be determined with due consideration of manufacturers specifications and recommended practices from professional organizations such as ITE.

The adjustment of maintenance levels by delaying needed rehabilitation will reduce expenditures to fit budget constraints. However, the negative safety consequences and societal costs of delaying needed maintenance should be weighed.

Intersection monitoring typically takes the form of routine data collection, periodic reviews of signal timing, investigation of public and police concerns, and regular assessment of intersection safety performance. Intersection turning movement counts and collision data are the primary sources of information required for monitoring intersection safety. Together, this data can be used to conduct a comparative assessment of the safety performance of the signals, and to determine if there is an increase in collision risk, or if the safety performance is unacceptable or aberrant.

Monitoring intersection efficiency is also an important safety consideration, since poor traffic operations may result in driver frustration and increased risk taking. Degrading traffic operations can be a precursor to future safety problems. Particular attention should be paid to vehicle detectors and pedestrian pushbutton maintenance, since failure of these devices causes unnecessary signal cycling and the actuation of phases that are subsequently unused because no vehicles or pedestrians are present. This results in driver, cyclist and pedestrian frustration as they experience unnecessary delays and stops, with the corresponding time loss plus vehicle emissions add excessive fuel consumption

If a safety performance issue is identified at an intersection, then an in service road safety review should be conducted. Current good practice for these reviews is provided in detail in the Canadian Guide to In-service Road Safety Reviews (TAC, 2004).

Quantitative Safety Aids

The safety impacts of various elements of signalized intersections have been quantified in many research studies, and a summary of the major findings are shown in Table 4.7, extracted from the ITE. This table is a good basic reference concerning the safety impacts of signal placement and design. However, the conventional wisdom concerning signal design and safety is constantly evolving, and the prudent practitioner will stay abreast of these developments by regularly reviewing literature, and by maintaining memberships in professional associations such as ITE and TAC.

Table 4.7 Safety impacts of signal operations and design

Numbers prior to the [n] represent the range of % crash reduction that might be expected from implementing a given improvement.

● **Countermeasure/Crash Type identified; however no estimate of effectiveness is provided.**

		Potential Effectiveness (Percentage Reduction)							
Improvement Type(s)	**Cost**	**Total Crashes**	**Right Angle Crashes**	**Left Turn Crashes**	**Rear-end Crashes**	**Sideswipe**	**Pedestrian**	**Red-Light Running**	**Older Driver**
SIGNAL OPERATIONS IMPROVEMENTS									
Interconnect/Coordinate Traffic Signals; Optimization	Medium	15-17 [1]	25-38 [12]		●			● [2]	
Increase/Modify Clearance Intervals	Low	4-31 [1,9,10]	1-30 [1,9]		●			● [2]	
Improve Signal Timing (General)	Low	10-15 [1]	●	●		●		●	
Add Protected/Permissive LT Phase	Medium	4-10 [1,9]		40-64 [1,9]					
Use Green Arrow/ Protected Left Turns/Movement Signal Phasing	Low	3 [9]		98 [9]					●
Use Split Phases	Low	25 [11]		●	●	●			
Use Leading Pedestrian Interval	Low						5 [8]		
Add Pedestrian Phase	Medium	23-25 [1]					7-60 [1,8]		
Add Left-Turn Phasing to an Existing Signal	Medium	23-48 [6, 12]		63-70 [1]			5 [8]		
Provide Green Extension (Advance Detection)	Variable				●			●	
Install Signal Actuation	Variable				●	●			
Assume Slower Walking Speeds for Pedestrian Signal Timing	Low						●		●
Provide Advance Warning of Signal Changes at Rural Signalized Intersections	Medium	●	●		●			●	
Remove Signals from Late Night/Early Morning Flash	Low	29[9]	80 [9]						
Consider Restricting Right-Turns-on-Red	Low						●		
Consider Installation of Pedestrian Countdown Signals (incremental cost)	Low						●		
Consider Installation of Animated Eye Signals (Incremental cost)	Low						●		
SIGNAL HARDWARE									
Install Larger (12-Inch) Signal Lenses	Low	10-12 [1,9]	48 [9]		●	●		●	●
Install Flashing Beacon at Intersection	Medium	30-38 [1]							
Install Flashing Beacon at Advance of Intersection	Medium	25-28 [1]						● [2]	
Replace Pedestal Mounted Signal with Mast Arm	High	28-43 [12]	●						
Install Backplates on Existing Signals	Low	2-24 [1,9]	7-93 [1,5,9]		●	●		● [2]	●
Optically Programmed Signal Lenses		15-18 [1]						●	
Provide Louvers, Visors, Special Lenses so Drivers are able to View Signals only for their Approach	Low				●	●		●	
Upgrade Signal Controller	Medium	20-22 [1 8, 11]		●	●	●			
Relocate/Shield Signal Hardware in Clear Zone. Signal Hardware Should Not Obstruct Sight Lines.	Medium	[6]	●		●	●			
Install Additional Signal Heads	Medium	10 [9]	42 [9]		●	●		●	●
Install More Overhead Traffic Signals	High	●	●		●			●	●
Provide Two Red-Signal Displays within each Signal Head to Increase Conspicuity of the Red Display	Medium							●[2]	
Use LED Traffic Signal Module.	Medium							● [2]	
Stripe for Left-Turn Lane within Existing Roadway	Low	26 [9]		66 [9]					
Red T-Display	Medium	9 [9]	36 [9]						

Table 4.7 Safety impacts of signal operations and design (continued)

		Potential Effectiveness (Percentage Reduction)							
Improvement Type(s)	Cost	Total Crashes	Right Angle Crashes	Left Turn Crashes	Rear-end Crashes	Sideswipe	Pedestrian	Red-Light Running	Older Driver
COMBINATION SIGNAL AND OTHER IMPROVEMENTS									
Construct Left-Turn Lanes with Signal Upgrades	High	●		●	●				
Left-Turn Lane, Signal and NO Turn Phase	High	21-25 [1]		46-54 [1]	●				
Left-Turn Lane, Signal PLUS Turn Phase	High	25-36 [1]		43-45 [1]	●				
Add Left-Turn Phasing AND Turn Lanes to an Existing Signal	High	46-69 [12]	●	●	●				
Removal Signal, Develop a Program to Identify and Remove Unwarranted Signals.	Low	50-53 [1]			●	●			
Install 12" Signal Heads and SIGNAL	Low	11 [9]	36 [9]						

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4.10.5 System Safety

Signalized intersections are part of the surface transportation network and, as such, the safety of individual signals may be impacted by system considerations. Typical system safety considerations include:

- Uniformity and Consistency: Consistent positioning of signal heads, and signal timing/phasing accommodates driver expectancies and promotes error-free driving.

- Signal coordination: The potential for rear-end and right-angle collisions is greatly reduced by the effective progression of traffic through successive signals.
- Vicinal Hazards: The dynamic displays at signalized intersections and the confluence of conflicting traffic streams place a high mental workload on road users. When other hazards, such as private driveways and horizontal curves, are placed within the functional area of the intersection, the attention of road users is diverted from the signal display. Drivers are serial processors of information, though their ability to handle one thing at a time usually in rapid succession and a confluence of significant hazards is apt to cause a driver to not notice one hazard because they are attending to a vicinal hazard.

4.10.6 Human Factors

Traffic signals are a part of the road-vehicle-human system in which the “human” is generally seen to be the weakest link. While it is difficult to modify human behaviour, through a better understanding of how humans react to the road environment, it is possible to predict how drivers and other road users will react to different features. Armed with this knowledge, it is then possible to incorporate aspects into the layout of signalized intersections that promote favorable responses and minimize potential driver errors.

Drivers anticipate common situations; these expectations are built from the upstream road environment as well as from past experience. Predictability in intersection layout, location and signal timing can reduce driver error. However, if intersections are not designed as expected or provide insufficient information, drivers can react in an erratic or unsafe manner.

Intersections in general should be designed such that they are expected by approaching drivers and provide sufficient information for vehicle operation and route choice decisions to be made without causing driver overload. Specific considerations with respect to signalized intersections include, but are not limited to, the following:

- Intersection location, layout and control type are consistent with expectations.
- Signal placement and timings are consistent with expectations.
- Signal timing does not result in excessive queues or delays that can cause driver frustration.
- Signal heads and signs are highly visible and located within the primary visual search area.
- Additional sight distances are provided for signals with complex or unique layouts.
- The intersection accommodates any special needs attributed to the expected road users, such as increased signal head size, or retro-reflective backboards in areas with high proportions of elderly drivers.

Positive guidance reviews are an application of human factors to the road environment, and are a useful tool in assessing or understanding the safety performance of a signalized intersection. A positive guidance review is an examination of the road or intersection from a road user's perspective that seeks to assess the adequacy of the information presented to the road user. The end goal is to ensure that the road system and the traffic control devices present information to the road user when the road user requires it, and in a format that is understandable so that error-free decisions can be made concerning speed and path of travel.

For a complete description of positive guidance reviews and related information, the reader is referred to Ontario Traffic Manual Book 1C - Positive Guidance Toolkit (MTO, 2001).

CHAPTER 5

SURVEYS

5.1 Arrival flow survey

The objective of the survey is to determine existing traffic demand for individual intersection movements.

5.1.1 Undersaturated conditions

Where only few signal cycles are overloaded, the arrival flow equals the departure flow across the stop line in directions L, T, R in Figure 5.1. The procedures and data presentation for such a standard intersection survey are described in many traffic engineering texts, manuals and handbooks.

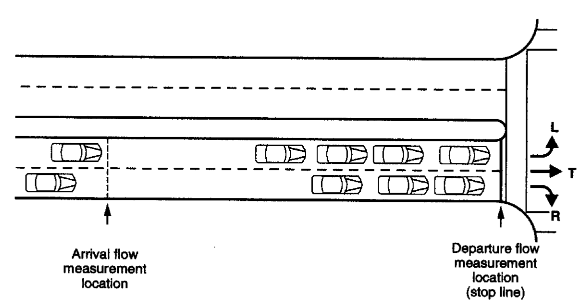

Figure 5.1 Measuring arrival and departure traffic flows.

5.1.2 Conditions at or over saturation

When vehicular traffic conditions are close to or over saturation, the probability of discharge overload is high, or a continuous overflow exists. The queues are long, and may be growing. The number of departing vehicles is constrained by capacity.

The standard survey of departure traffic flows must be supplemented by a simultaneous count of vehicles arriving at the end of the queue (arrival flow measurement location in Figure 5.1). An additional survey person at the point beyond the reach of the queue on the overloaded approach may be needed.

In some instances, the intersection approach lane configuration and the length of the queues do not allow an identification of the directions in which the vehicles arriving at the end of the queue (queues) will be discharging. The arrival flow in individual intersection directions are therefore prorated in relationship to the discharge flows obtained at the stop line. In critical cases, a license plate origin-destination survey at the end of a queue (origin) and at the intersection exits (destinations L, T, R in Figure 5.1) is advisable.

5.2 Saturation flow survey

The objective of the survey is to determine the maximum rate of discharge across a stop line of a given lane during the green interval.

5.2.1 Field work and notes

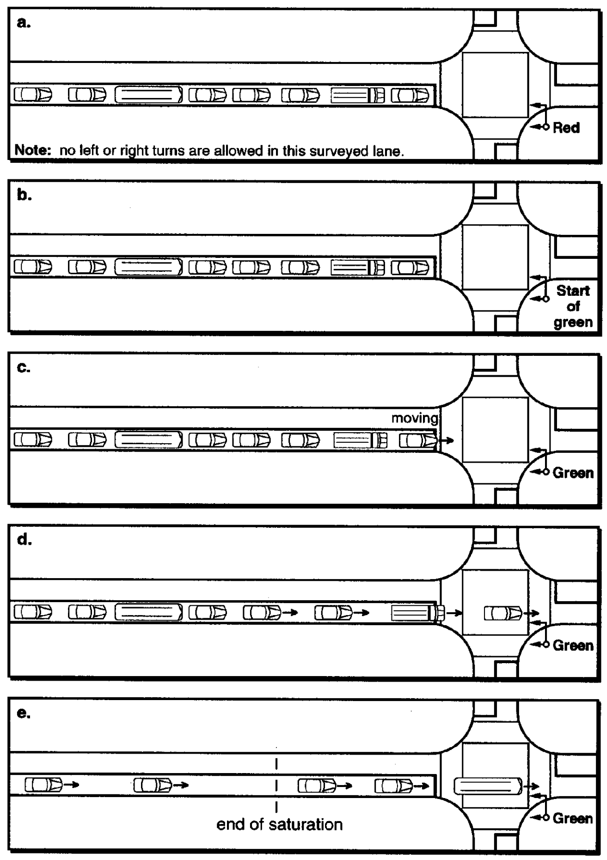

Figure 5.2 Saturation flow survey activities.

a. One person with a tape recorder:

All events are dictated in the field and replayed in the office. Data are entered onto the sheets in the same way as for a two-person survey under (b) below. Since the necessary time information from the survey location is kept on the tape and measured in the office, a field and office time consistency test of about 15 s is advisable.

b. Two people:

The first person acts as observer who dictates information; the second person records and acts as timekeeper. With practice, two people can survey and record two lanes simultaneously.

The field form is shown in Table 5.1, and an example of the field notes in Table 5.2. See pages 5-128 and 5-129.

The series of events to be recorded is shown in Figure 5.2. The survey instructions are as follows:

1. Identify and note intersection name, approach direction, lane, time, weather, special conditions (if any exist), and surveyors' names. Include a sketch illustrating the direction and the surveyed lane.

2. Before the start of green (when amber in the previous phase appears), the observer identifies the number of vehicles in queue. For instance, “Q8”, noted by the recorder in the queue column. This value is not used directly; only to verify that a sufficient number of vehicles has been accumulated to sustain saturation flow (Figure 5.2a).

3. The observer says “G” at the instant when the green signal appears (exactly at that moment). The recorder/timekeeper starts the stopwatch (Figure 5.2b).

4. When the *front bumper* of the first vehicle crosses the *stop line* the observer says: “C” (for a car) (Figure 5.2c), “T” (for a truck), or “B” (for a bus), etc., depending on vehicle classification used.

5. When the front bumper of the next vehicle (and of the following vehicles) reaches the stop line, the vehicles are identified in the same manner as for the first vehicle (Figure 5.2d).

6. The recorder/timekeeper writes down the letters dictated by the observer. He/she also keeps the stopwatch in the field of vision. After every five seconds, a new column is started, or a vertical line is drawn.

7. The observer also visually follows the vehicles that are joining the end of the queue and announces “End of saturation” when the continuous maximum flow at the stop line has ended (Figure 5.2e). Typically, for straight-through, protected left-turn or unconstrained right-turn movements, saturation flow is no more available when two consecutive five-second intervals contain less than two passenger car units, terminating at the end of the last time increments with two or more passenger car units. The identification of the end of saturation in other cases is guided by the presence - or absence - of waiting vehicles. The recorder draws a vertical line after the last vehicle to clearly designate that the saturation flow in this cycle ended. A horizontal line is drawn through those green interval increments in which no vehicles departed.

8. The survey should include at least 30 cycles with full saturation during the first 20 to 30 s. Exceptionally, in smaller communities, 20 cycles with 10 to 20 s of saturation may suffice to estimate an approximate saturation flow as shown in Figure 5.4 on page 5-132.

9. Additional information may be included. For example, as shown in Table 5.2, the count of the departures may continue after the saturated portion of the green interval and into the amber interval, and possibly, during the red interval for right-turn lanes with RTOR.

10. Although not needed for the saturation flow investigation, useful data include the number of vehicles that were unable to depart during the green and amber interval. These vehicles are shown in Table 5.2 and Table 5.3 in the last two columns. Note that although some of the cycles feature vehicles that crossed the stop line or stopped during the amber interval, they were not oversaturated. This information serves for the determination of the overload factor (See 5.4 “Overload Factor survey” on page 5-135).

Table 5.1 Example of a saturation flow survey form

Day	Name	Location	Direction	Sketch
Start time	Notes			

Cycle #	**Queue at start of green**	**Vehicles in 5-second increments of green interval**								**Departures during amber**	**Queue at end of amber (vehicles)**
		0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40		
1											
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
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16											
17											
18											
19											
20											
21											
22											
23											
24											
25											
26											
27											
28											
29											
30											

Table 5.2 Example of saturation flow survey notes

Cycle #	Queue at start of green	pcu in 5-second increments of green interval								Departures during amber	Queue at end of amber (vehicles)
		0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40		
1	9	cc	ccc	ccc	cc						
2	15	c	cc	cc	cc	ccc	ccc	cc	cc		1
3	18	cc	cc	cc	ccc	cc	cccc	ccc			1
4	9	c	cccc	T	ccc	c	cc			1	
5	11	cc	cc	ccc	cc	ccc	ccc				
6	16	cc	ccc	cc	Tc	ccc	cc	ccc	cc	1	2
7	17	B	cc	cccc	ccc	cc	ccc	cc	ccc		
8	11	cc	cc	ccc	T	cc					
9	17	cc	cc	cc	cccc	cc	cc	ccc	cc		
10	7	cc	c T	ccc							
11	16	cc	ccc	cc	cc	ccc	c T	c	ccc	1	1
12	15	c	cc	ccc	cc	ccc	ccc	cc	ccc	1	3
13	17	cc	ccc	cccc	cc	c Tc	cc	cccc	c	2	2
14	14	cc	T cc	cc	ccc	ccc	cc	cc	ccc	1	2
15	15	cc	cc	ccc	ccc	cc	ccc	ccc	cc	1	1
16	18	T c	cc	c T	cccc	ccc	cc	ccc	cc	1	3
17	13	c	ccc	cc	cc	ccc	cc	ccc			
18	6	cc	cc	ccc							
19	14	cc T	c	cccc	ccc	cc	ccc				
20	6	cc	c T	cc	cc						
21	13	cc	ccc	cc	ccc	cc	cc	T c	ccc		1
22	10	cc	ccc	c T	cc	ccc	ccc				
23	14	cc	ccc	cc	cccc	ccc	c	cc	ccc		2
24	11	c	B	cc	ccc	c T	ccc	ccc			
25	6	T	cc	ccc	cc						
26	4	cc	ccc	cc							2
27	12	cc	cc	cc T	ccc	cc	ccc	cc			
28	4	T	ccc	cc							
29	14	cc	ccc	T c	ccc	cc	c T	cc	ccc	1	
30	9	cc	cc	ccc	cc	cc	ccc			1	
31	3	ccc	c	cc							
32	13	cc	cc	ccc	cc	ccc	c T c	cc			

5.2.2 Transcript of data from the field notes

An example, based on the field notes from Table 5.2, is shown in Table 5.3. The numbers in the portions of the green intervals are converted to passenger car units, using the passenger car equivalents for individual vehicle categories from Table 3.2 "Passenger car unit equivalents" on page 3-15.

Table 5.3 Example of a transcript of the saturation flow field notes (data from Table 5.2)

Cycle #	Queue at start of green	pcu in increments of green interval (pcu/5 s)								Departures during amber	Queue at end of amber (vehicles)
		0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40		
1	8	2	3	3	2						
2	15	1	2	2	2	3	3	2	2		1
3	18	2	2	2	3	2	4	3			1
4	9	1	4	1.5	3	1	2			1	
5	11	2	2	3	2	3	3				
6	16	2	3	2	2.5	3	2	3	2	1	2
7	17	2	2	4	3	2	3	2	3		
8	11	2	2	3	1.5	2					
9	17	2	2	2	4	2	2	3	2		
10	7	2	2.5	3							
11	16	2	3	2	2	3	2.5	1	3	1	1
12	15	1	2	3	2	3	3	2	3	1	3
13	17	2	3	4	2	3.5	2	4	1	2	2
14	14	2	3.5	2	3	3	2	2	3	1	2
15	15	2	2	3	3	2	3	3	2	1	1
16	18	2.5	2	2.5	4	3	2	3	2	1	3
17	13	1	3	2	2	3	2	3			
18	6	2	2	3							
19	14	3.5	1	4	3	2	3				
20	6	2	2.5	2	2						
21	13	2	3	2	3	2	2	2.5	3		1
22	10	2	3	2.5	2	3	3				
23	14	2	3	2	4	3	1	2	3		2
24	11	1	2	2	3	2.5	3	3			
25	6	1.5	2	3	2						
26	4	2	3	2							2
27	12	2	2	3.5	3	2	3	2			
28	4	1.5	3	2							
29	14	2	3	2.5	3	2	2.5	2	3	1	
30	9	2	2	3	2	2	3			1	
31	3	3	1	2							
32	13	2	2	3	2	3	3.5	2			
V_s		61	77.5	82.50	70.0	60.0	59.5	44.5	32		
n_s		32	32	32	27	24	23	18	13		

5.3 Calculations

5.3.1 Saturation flow in increments of the green interval

The summary of each column in Table 5.3 includes the number of passenger car units in all valid saturated increments of green intervals, and the number of these valid portions.

The saturation flow in each portion of the green interval is then determined as follows:

$S_i = 3600 / h$

or, by substituting for

$h = t_g n_s / V_s$:

$S_i = 3600 V_s / t_g n_s$

where:

S_i = saturation flow in a given increment of the green interval (pcu/h)

h = average saturation headway (s)

V_s = total number of passenger car units in all saturated portions of green intervals (column total)

t_g = duration of the green interval increments (5 s, or exceptionally 10 s)

n_s = number of fully saturated increments of green intervals.

The value of the saturation flow for each portion of the green interval is determined for all columns. Where the number of valid portions of the green intervals is below 20, the result may be statistically not significant. Less than 10 valid portions of the green intervals should not be used.

The summary calculation is shown in Table 5.4.

Table 5.4 Determination of saturation flows in individual increments of the green interval

Cycle #	Queue at start of green	Green interval increments (5 s)								Departures during amber	Queue at end of amber (vehicles)
		0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40		
V_s		61	77.5	82.50	70.0	60.0	59.5	44.5	32		10
n_s		32	32	32	27	24	23	18	13	32	32
h		2.62	2.06	1.94	1.93	2.00	1.93	2.02	2.03	N.A.	
S_i		1373	1744	1856	1867	1800	1863	1780	1772	N.A.	

5.3.2 Canadian Capacity Guide method

a. Simple average

If the valid portions of the green interval include more than 25 s, a simple average may be used as a conservative estimate of the saturation flow. In the example in Table 5.4:

$S_{average}$ = (1373 + 1744 + 1856 + 1867 + 1800 + 1863 + 1780 + 1772) / 8 = 1756.9 pcu/h

The last saturated flow value may be considered statistically somewhat less reliable because the sample size of only 13 saturated increments of the green interval.

In order not to create an impression of high precision, the resulting values should be rounded to 5 or 10 pcu/h:

S_{aver} = 1756.9 pcu/h ≅ 1755 pcu/h

Table 5.5 Summary of saturation flows in individual increments of the green interval (based on Table 5.4)

Cycle #	Saturation flow in green interval increments (pcu/h green)							
	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40
S_i	1373	1744	1856	1867	1800	1863	1780	1772

b. Cumulative average

The presentation of the results in a cumulative average graph is advisable in all instances because the graph provides additional insight into driver behaviour at the beginning of the green interval. It may also help reveal problems in statistical validity.

Using the example included in Table 5.3 and Table 5.4, the individual points of the cumulative average diagram are calculated as follows:

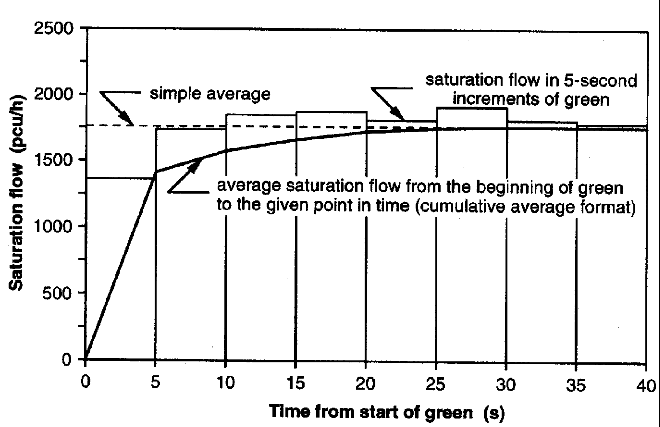


Figure 5.3 A comparison of the simple average and cumulative average representation of the saturation flow survey results from Table 5.6.

S_{0-5} = 1373 pcu/h

S_{0-10} = (1373 + 1744) / 2 = 1539 pcu/h

S_{0-15} = (1373 + 1744 + 1856) / 3 = 1658 pcu/h

S_{0-20} = (1373 + 1744 + 1856 + 1867) / 4 = 1710 pcu/h

S_{0-25} = (1373 + 1744 + 1856 + 1867 + 1800) / 5 = 1728 pcu/h

S_{0-30} = (1373 + 1744 + 1856 + 1867 + 1800 + 1863) / 6 = 1751 pcu/h

S_{0-35} = (1373 + 1744 + 1856 + 1867 + 1800 + 1863 +1780) / 7 = 1754 pcu/h

S_{0-40} = (1373 + 1744 + 1856 + 1867 + 1800 + 1863 +1780 + 1772) / 8 = 1757 pcu/h

The resulting cumulative average values are shown in Table 5.6 and Figure 5.3. The saturation flow value is estimated from the asymptotic horizontal value in the graph. In the example, the saturation flow estimated using Figure 5.3 is approximately:

$S_{\Sigma i}$ = 1755 pcu/h

Since the surveyed green interval in this example was often saturated until the beginning of the amber interval at 40 s after the green start, this asymptotic value coincides with the simple average. If only a shorter portion were available for the measurement because of the lack of fully saturated increments, say, 25 s, the simple average saturation flow would be its $S_{0\text{-}25}$ value of 1728 pcu/h. The rising curve in Figure 5.3 indicates that the asymptotic value of the saturation flow curve is higher and, extrapolating the trend in the graph, would be estimated at about 1750 pcu/h. The difference in comparison to the simple average becomes smaller with the increasing number of the valid increments of the green interval.

The consistency of the slope of the cumulative average saturation flow curve is typical for statistically valid samples. This appears to be the case even for the value in the last increment and suggests that the full sample may be used, despite the somewhat smaller number of saturated green increments in the range of 35 to 40 s.

The cumulative format is especially useful for saturation flow estimation at intersections in smaller communities. At these locations, the valid saturated increments of the green interval often do not exceed 20 s because of low arrival flows and short queues. In these instances, the 5-second portions of the green intervals allow the plotting of local values in the diagram of saturation flows for the five cities shown in Figure 3.8 "Typical measured saturation flows in Canadian cities in the cumulative average format." on page 3-30. The trend revealed by these plotted points is then extrapolated between or below the other lines to determine an approximate highest value that represents the estimated local saturation flow. In Figure 5.4, the saturation flow values for the first three time increments have been placed in the diagram and the remaining values extrapolated to demonstrate the process.

Table 5.6 A comparison of saturation flows in individual increments of the green interval with the cumulative average saturation flow values calculated from the start of the green interval

Saturation flow type	Saturation Flow (5pcu/h)							
	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40
in individual increments (S_i)	1367	1738	1856	1867	1800	1863	1780	1772
average over green interval (S_{aver})	1757							
cumulative average ($S_{\Sigma i}$)	1373	1559	1658	1710	1728	1751	1755	1757

The extension of the trend revealed by the three sets of survey data indicate a saturation flow of about 1750 pcu/h. This is similar to the value determined on the basis of the full survey sample.

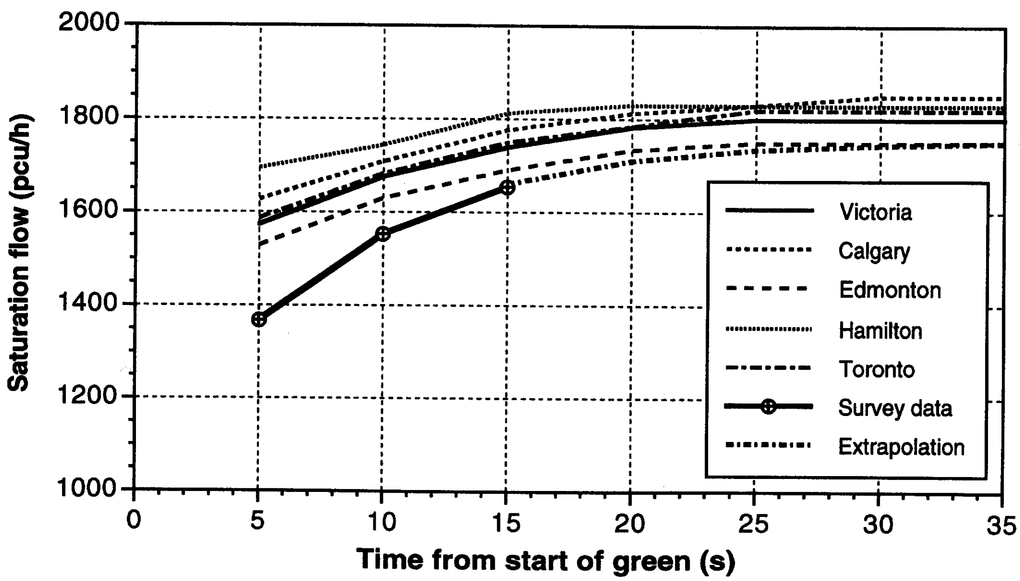


Figure 5.4 Estimation of the saturation flow for situations where only a limited number of the green interval increments can be surveyed.

Highway Capacity Manual method (HCM 2000)

The values from this example survey should not be used directly. They are not applicable, since the *HCM surveys start with the fourth vehicle after the beginning of the green interval* and, in this case, apparently begins prior to the end of the second time increment. The HCM saturation flow, however, may be estimated from the saturation flow determined by the Canadian method using the following approximate relationship ("Relationship between Saturation Flow in this Guide and in the Highway Capacity Manual (HCM)" on page 3-26):

$S_{HCM} = 1.05\ S_{CCG}$

where:

S_{HCM} = saturation flow for the use in the Highway Capacity Manual average stopped delay formula (pcphgpl = passenger cars per hour green per lane)

S_{CCG} = saturation flow measured and calculated as described here (pcu/h).

In the example above:

$S_{HCM} = 1.05\ S_{CCG} = 1.05 \times 1757 = 1844.9 \cong 1845$ pcphgpl

Australian method (Akcelik 1981)

The survey includes vehicles that crossed the stop line *after 10 seconds* of the green interval. The saturation flow value is determined from the average saturation headway for the period starting at the tenth second after the beginning of the green interval. In the example in Table 5.4, the saturation flow may be determined here as the number of seconds in an hour, divided by the average saturation headway after the first 10 seconds of the green interval. This average of all saturated headways is calculated as the average headway in each increment weighted by the number of passenger car units in each increment and divided by the total number of passenger car units during all increments included:

S_{AUS} = 3600 / [(1.94 x 82.5 + 1.93 x 70 x 2.00 x 60 + 1.93 x 59.5 + 2.02 x 44.5 + 2.03 x 32) / (82.5 + 70 + 60 + 59.5 + 44.5 + 32)] $\cong$ 1832 pcu/h

The saturation flow value determined by the Australian survey method is usually similar to the Highway Capacity Manual value or slightly higher. In this example, the difference between the approximate Australian and approximate HCM saturation flows is less than 1%.

5.3.3 Direct determination of capacity and effective green interval

Capacity of the green interval

The capacity of a given green interval may be determined from its duration and the average headway. Alternatively, using the additional survey data, it can be calculated as the average number of passenger car units that discharged during fully saturated or oversaturated cycles.

a. Determination of the green interval capacity from average headway

Capacity of the green interval can be determined from the following relationship:

Xg = ge / h = ge S / 3600

where:

X_g = capacity of the green interval (pcu/green interval)

g_e = g + 1 = effective green interval (s)

g = displayed green interval (s)

h = average headway (s)

S = measured saturation flow (pcu/h).

In our case:

X_g = (40 + 1) 1757 / 3600 = 20.10 pcu/green interval

b. Determination of the green interval capacity from average discharge in fully saturated or oversaturated cycles

In Table 5.2 and Table 5.3, green intervals in cycles number 2, 6, 11, 12, 13, 14, 15, 16, 21 and 23 were identified as overloaded, having residual queues at the end of the amber interval. In addition, green intervals in cycles number 7, 9 and 29 were also apparently fully utilized. The total number of fully saturated or oversaturated cycles is therefore 13.

From Table 5.3, the sum of all passenger car units that discharged in these fully saturated or oversaturated cycles is 265.5. The average capacity of these green intervals is therefore:

X_g = 265.5 / 13 = 20.42 pcu/green interval

This value is close to that calculated using the usual effective green interval indicated in "Green interval and effective green interval" on page 3-58.

Effective green interval

In the first capacity calculation, the effective green interval was assumed to be longer than the displayed green interval by one second. This assumptions may be tested using the measured saturation flow value and the capacity determined from fully saturated or oversaturated green intervals. Since:

$X_g = g_e S / 3600$

and the capacity of the green interval X_g and saturation flow S are known,

$g_e = 3600 X_g / S.$

In our case:

$g_e = 3600 \times 20.42 / 1757 = 41.839$ s

This indicates that the actual effective green interval is longer by 1.84 s than the displayed green interval. This finding is consistent with the note in "Green interval and effective green interval" on page 3-58, suggesting that the usual value of = g + 1 is somewhat conservative for the measured conditions.

Hourly capacity

In order to determine the hourly capacity of the surveyed lane, the cycle time and the discharge during periods other than the green interval must be known. In our case, the cycle time was 100 s. The surveyed lane carried only straight-through traffic flow and no departures on red took place. Therefore (as per Section 4.7.1 "Capacity of approach lanes for vehicular traffic" on page 4-96):

$C = S\, g_e / c = 1757 \times 41.839 / 100 = 735.1$ (rounded to nearest 10) $\cong 740$ pcu/h

Using the green interval capacity established from the fully saturated or oversaturated cycles, the hourly capacity would be a product of the green interval capacity and the number of cycles in an hour:

$C = 3600 X_g / c = 3600 \times 20.42 / 100 = 735.1$ (rounded to nearest 10) $\cong 740$ pcu/h

The results from the two different equations are the same.

5.4 Overload Factor survey

The objective of this survey is to determine the proportion of cycles in which the accumulated demand exceeds the capacity of a given lane.

The survey concentrates on determining whether the green interval was continuously utilized at the saturation flow level, and whether any drivers who arrived in a cycle were unable to discharge during the same cycle. Cycles with stalled vehicles, or any other unusual events, and the subsequent cycles with follow-up overloads are not considered.

The survey should include a minimum of 20 *consecutive* cycles.

In an oversaturated lane with a growing queue, the survey would yield an overload factor of 1.0 and is therefore not needed.

5.4.1 Field work and notes

Approximate method

The surveyor starts with an identification of the conditions. There should be no overflow queues at beginning of the survey. An example of a simple field form is shown in Table 5.7.

Table 5.7 A simplified overload factor survey form

Cycle #	Number of vehicles in queue at the beginning of green	Green interval has been fully saturated yes or no	Queue at the end of amber; if > 0 and green was fully saturated, the cycle was overloaded
1	9	no	0
2	15	yes	1
3	18	no	1
4	9	no	0
5			
6		etc.	
7			
etc.			
n	N.A.		n_o

Detailed survey

This survey may be combined with a saturation flow survey. The saturation flow survey procedure described in Section 5.2 "Saturation flow survey" on page 5-124 is then used. The form is shown in Table 5.1 on page 5-126. The overloaded cycles are identified as the cycles with at least one vehicle in a queue at the end of the amber interval, and with uninterrupted discharge at saturation. The average discharge in these cycles can also be used *to verify the calculated value of capacity* and, by back calculation, for *the determination of a local value of the effective green interval* ("Green interval and effective green interval" on page 3-58).

5.4.2 Calculations

The overload factor is calculated as

$$OF = n_o / n$$

where:

OF = overload factor

n_o = number of overloaded cycles

n = total number of consecutively surveyed cycles, with $n \geq 20$.

Note that in the example shown in Table 5.2 on page 5-127 and Table 5.3 on page 5-128, cycles number 7, 9 and 29 were fully saturated but did not feature an overload because no leftover queues were recorded. Queues at the end of the amber interval in cycles number 3 and 26 were not an indication of an overload because these cycles were not utilized at full saturation in the last increments of the green interval. The number of truly overloaded cycles was only 10. The overload factor therefore is:

$$OF = 10/32 \cong 0.31 = 31\%$$

In Table 5.2, the total volume during the surveyed period of 32 cycles of 100 s each was 487 pcu. This corresponds to an arrival flow of 548 pcu/h or approximately 15 pcu/cycle. The cycle capacity, calculated from the green interval 40 s with the effective green interval 41 s and saturation flow of 1755 pcu/h was approximately 20 pcu/cycle.

Therefore:

m = 15 pcu/cycle

X_c = 20 pcu/cycle

Interpolation in Figure 4.3 "Probability of discharge overload." on page 4-98, indicates that the probability of discharge overload is about 18% in this case where the average cycle capacity is 20 pcu/cycle and the average number of arrivals is 15 per cycle. This value is lower than the overload factor observed. The 10-minute traffic surge in cycles 11 to 16 had apparently some influence. It should also be remembered that while the measured overload factor is subject to random variations in consecutive groups of cycles with the same arrival flow, the probability of discharge overload represents an average or a typical value. For that reason, the once only measured overload factor will rarely be in complete agreement with the calculated probability of discharge overload. For research investigations, the surveys must be repeated several times under identical conditions, or an adequate computer simulation program applied.

If the simplified survey procedure were used, the number of overloaded cycles would also be 10. It is important that those cycles that had a queue at the end of amber but were not continuously loaded to saturation, are not counted as cycles with overflow.

5.5 Average overall delay survey

The objective of the survey is to determine the average overall delay considering all vehicles discharging across the stop line of the surveyed lane. The survey should cover the whole analysis period (3.1.5 "Analysis period, evaluation time, design period, period of congestion, and transit assessment time" on page 3-17**) or evaluation time used in the delay formula (**"Average overall lane delay in s/pcu" on page 4-101**). The recommended minimum duration is 15 minutes.**

5.5.1 Field work and notes

Normally, two surveyors are involved. The first surveyor is stationed at a point upstream of the intersection, beyond the reach of the queues; the second surveyor is at the stop line. The first surveyor measures the arrival flow, the second surveyor measures the departure flow. Both measurements are coordinated in time, in survey intervals of the same duration, normally 10 seconds. In very low traffic conditions, a shorter measurement interval is appropriate. Flows at both survey locations are identified in their individual vehicle categories and noted at the end of the survey interval.

Preferably, the survey starts with no overflow queue at the beginning of the first surveyed cycle, designated as the beginning of the first red interval. If this situation cannot be avoided, the number of queued vehicles is added to the number of vehicles *arriving* during the first measurement interval.

The free-flow cruising travel time between the two measurement points can be determined as the average travel time of vehicles that proceeded over the intersection approach and across the stop line unimpeded. Test runs may also be used, or the average cruising travel time estimated from the distance between the measurement point of the arrival flow and the stop line, divided by an appropriate speed.

5.5.2 Calculations

First, the arrival and departure flows are converted to flows in pcu/h (Table 3.2 on page 3-15). Then, the average overall delay is determined as the total overall delay to all vehicles that arrived during the analysis period, divided by the number of vehicles that discharged during that period. *The average free-flow travel time between the two measurement points must be subtracted.*

Normally, at least 15 minutes of steady traffic conditions should be surveyed.

Average overall delay is determined as:

$$d = [t_m \Sigma (\Sigma X_{i\ arr} - \Sigma X_{i\ dep}) / \Sigma X_{i\ dep}] - t_t$$

where:

d = average overall delay (s/pcu)

t_m = survey interval (s), typically 10 to 30 s

$X_{i\ arr}$ = arriving passenger car units during survey interval i (pcu)

$X_{i\ dep}$ = departing passenger car units during survey interval i (pcu)

$\Sigma X_{i\ arr}$ = cumulative sum of arriving passenger car units per survey interval (pcu)

$\Sigma X_{i\ dep}$ = cumulative sum of departing passenger car units per survey interval (pcu)

$\Sigma (\Sigma X_{i\ arr} - \Sigma X_{i\ dep})$ = sum of the differences between the number of passenger car units that have arrived or departed from the beginning of the survey up to the end of survey interval i. In Table 5.8, this value is determined for each row.

t_t = cruising travel time over the distance between the point where the arrival flow is measured and the stop line (s).

A simple numerical example is shown in Table 5.8. It includes only two cycles of 100s each, employs a measurement interval of 10 s, the distance of 70 m separating the upstream survey location from the stop line, and the speed limit is 50 km/h. The example shows one lane only. Lanes with similar saturation flows may be combined.

Table 5.8 Simplified example of an overall delay survey calculation

Time(s)	Number of vehicles arriving $X_{i\ arr}$	Number of vehicles departing $X_{i\ dep}$	Sum of vehicles arriving $\Sigma X_{i\ arr}$	Sum of vehicles departing $\Sigma X_{i\ dep}$	Difference of sums arriving and departing $\Sigma X_{i\ arr} - \Sigma X_{i\ dep}$
10	3	0	3	0	3
20	4	0	7	0	7
30	2	0	9	0	9
40	1	0	10	0	10
50	2	0	12	0	12
60	3	4	15	4	11
70	1	5	16	9	7
80	0	5	16	14	2
90	3	4	19	18	1
100	2	2	21	20	1
110	3	1	24	21	3
120	2	0	26	21	5
130	0	0	26	21	5
140	3	0	29	21	8
150	1	0	30	21	9
160	2	3	32	24	8
170	3	5	35	29	6
180	2	6	37	35	2
190	4	3	41	38	3
200	2	2	43	40	3
Totals	43	40	N.A.	N.A.	115

In the example above, the overall delay is:

$d = (10 \times 115 / 40) - 70 / (50/3.6) = 23.71$ s/pcu

5.6 Queue reach survey

The objective of this survey is to determine the maximum probable queue reach or a complete distribution of queue reaches during the analysis period.

5.6.1 Field work and notes

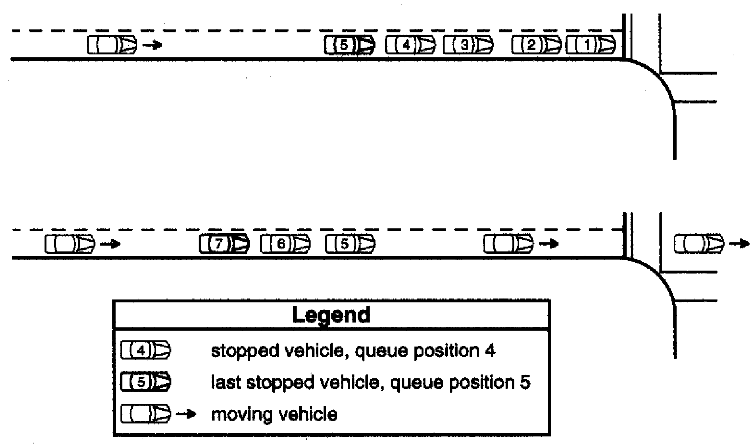


Figure 5.5 Examples of queue position count in a queue reach survey. Note that in the second illustration, the maximum queue reach is queue position 7

The surveyor selects a location with a good view of the end of the queued vehicles and of the corresponding signal head. The position of the vehicle stopped the farthest from the stop line as the queue increases and decreases is noted with each vehicle joining the end of the queue. This situation represents the queue reach. The vehicles at the front of the queue may be moving.

The queue reach includes the distance between the stop line and the front of a stationary queue expressed in the number of vehicles, plus all stopped vehicles, plus those vehicles that are in the process of stopping. Walking speed is a good approximate criterion for a vehicle that should be included. The situation is shown in Figure 5.5.

At the end of each cycle, a new line in the survey form is started. An example of the queue reach survey form is shown in Table 5.9, and an example of the notes for a left-turn lane in Table 5.9. The numbers represent the position of the last vehicle in the queue counted from the stop line. The count includes positions that are no more occupied by stopped vehicles because the front of the queue has already dissipated.

The survey should include at least 20 cycles.

Table 5.9 Example of a queue reach survey form

Day	**Name**	**Location**	**Direction**	**Sketch**
Start time	**Notes**			

Cycle #	**Position of the vehicle at the end of the queue**										**Queue Reach**

5.6.2 Calculations

In each of the surveyed cycles, the longest queue reach is identified and underlined as shown in Table 5.10. This situation is characterized by the greatest number in the row. The objective of the calculation is to determine the relative frequency of the longest queue reaches.

Table 5.10 Example of a queue reach survey (a left-turn lane)

Cycle #	Position of the vehicle at the end of the queue[1]	Queue reach
1	0 1 2 3 4 5 <u>6</u> 4 3 4 2 0	6
2	0 1 3 4 5 <u>8</u> 6 5 7 4 2 1 0	8
3	0 2 3 4 6 7 8 9 10 <u>11</u> 9 8 7 5 4 3 2 2	11
4	2 3 4 5 7 8 <u>9</u> 8 7 5 4 2 0	9
5	0 2 3 4 5 6 <u>8</u> 7 6 5 6 5 4 3 2 0	8
6	0 1 2 3 <u>4</u> 3 4 3 2 1 0	4
7	0 1 2 3 5 <u>6</u> 5 6 5 4 3 4 2 1 0	6
8	0 2 3 4 6 8 9 <u>10</u> 9 8 6 5 3 2 1 0	10
9	0 2 3 5 6 7 9 10 11 <u>13</u> 12 10 9 8 9 7 6 5 4 3	13
10	3 4 6 7 <u>9</u> 8 7 6 5 4 5 3 2 1 0	9
11	0 2 4 5 6 8 <u>9</u> 8 7 6 7 6 4 3 2 1 2	9
12	2 3 5 6 7 8 10 11 <u>12</u> 11 10 12 10 9 8 6 5 4 3 2 1	12
13	1 3 4 5 6 7 8 <u>9</u> 7 6 5 7 6 5 4 3 2 1 0	9

Table 5.10 Example of a queue reach survey (a left-turn lane)

Cycle #	Position of the vehicle at the end of the queue[1]	Queue reach
14	0 1 2 3 5 6 5 4 3 2 3 4 2 1 0	6
15	0 1 3 4 5 4 3 2 3 4 3 2 1 0	5
16	0 2 3 4 5 7 8 9 10 9 8 7 6 8 7 6 5 4 3 4 3 2 0	10
17	0 1 2 3 5 6 7 8 9 8 7 8 6 5 4 5 3 4 2 1 0	9
18	0 2 3 4 6 7 8 7 6 7 5 4 3 4 2 1 2 0	8
19	0 1 3 4 5 6 7 8 10 9 8 7 6 8 7 6 5 6 5 4 3 4 2 1 0	10
20	0 2 3 4 3 4 3 4 3 4 3 2 0	4

1. The positions may be converted to passenger car units

The relative frequency, equated approximately with the probability of a selected queue reach, is determined as:

$P(Q_i > Q) = \Sigma\, n_{(Qi > Q)} / n$

where:

Q_i = queue reach in any cycle (veh or pcu)

Q = selected queue reach (veh or pcu)

$P(Q_i > Q)$ = probability of a queue reach exceeding the selected queue reach Q

$\Sigma\, n_{(Qi > Q)}$ = number of cycles with the queue reach exceeding the selected queue reach Q

n = total number of cycles surveyed.

The farthest queue reach in each cycle is determined as the greatest number that describes the position of the end of the queue. This number is identified in the last column.

All of these numbers are then sorted and their relative frequency is determined as illustrated in Table 5.11

Table 5.11 Example of the determination of the queue reach frequency (left-turn lane example)

Farthest queue reach in each cycle Q (veh)	3	4	5	6	7	8	9	10	11	12	13	14
Frequency	0	2	1	4	0	2	5	3	1	0	2	0
Relative frequency (%)	0	10	5	20	0	10	25	15	5	0	10	0
Cumulative frequency $P(Q_i > Q)$ (%)	100	90	85	65	65	55	30	15	10	10	0	0

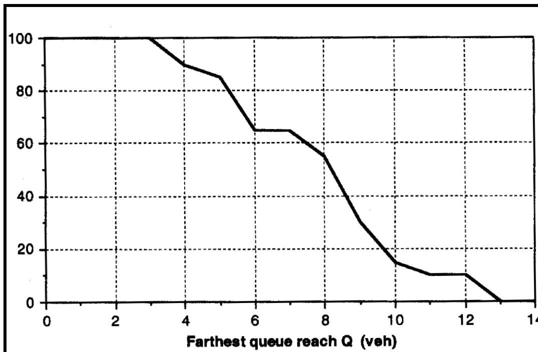


Figure 5.6 Distribution of queue reach data from Table 5.11.

Figure 5.6 illustrates the cumulative frequency.

In this instance, since 20 cycles were surveyed and the maximum queue reach of 13 was observed twice, the probability of a queue reach exceeding 12 is:

$P(Q_i > 12) = 2/20 = 0.10 = 10\%$

This value may also be read from the graph in Figure 5.6. It indicates that the likelihood of queue reach longer than this observed maximum is relatively low. Similarly, the probability of a queue exceeding, say, 9, determined from Figure 5.6 or directly from Table 5.11 is 30%. These values may be compared with the probability of maximum queue reach calculated using the equation in 4.8.4 "Vehicular queues" on page 4-108, or read from the graph in Figure 4.7 on page 4-111 in that section. Nevertheless, while the equation and the graph represent typical average conditions, the survey yields only one point in the distribution of all possible outcomes measured under identical traffic flow and conditions.

The duration of the survey depends on sustained steady flow conditions. The longer they last, the longer the survey can be. If a more detailed analysis is desired, at least 30 cycles should be included in the survey. The results can be plotted in a relative frequency diagram similar to Figure 5.6 on page 5-142. Any required percentile, including the median, can then be determined. The median may be used to approximate the queue reach and compared with the liberal or conservative average queue reach determined by the procedures in 4.8.3 "Number of stops" on page 4-107.

CHAPTER

6

THE PROCESS: EXAMPLES

6.1 Worked Example 1

The purpose of this example is to demonstrate the essence of the three principal steps in the process: *analysis, design and evaluation*, and the relationship among them. Since a thorough understanding of the basic process is essential, a highly simplified intersection situation is used in order not to pose any complications to the computational sequence.

A related goal is to give an example of a useful basic computational format that can be expanded as required by the complexity of the task at hand. All tasks related to a signalized intersection, no matter how complicated the layout and cycle structure are, may employ similar format. Additional considerations and computations are inserted as needed.

6.1.1 The design problem

The task is to design the signal timing for a simple isolated intersection with four one-lane approaches. No turning movements are allowed. The speed limit on all approaches is 50 km/h. The geometric conditions at the intersection are shown in Figure 6.1.1 .

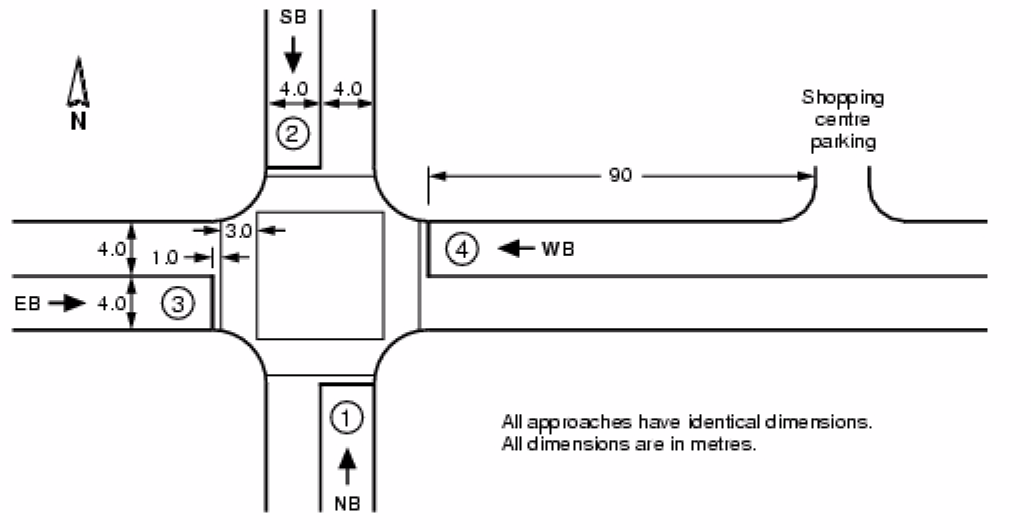


Figure 6.1.1 Intersection layout for the example

6.1.2 Analysis

The analytical part of the task requires the identification of evaluation period, arrival flows and pedestrian flows, saturation flows, tentative cycle structure and the associated intergreen periods and pedestrian requirements.

Analysis period, evaluation time and transit assessment time

In this example, the signal timing design will be calculated for a p.m. peak period of a typical workday. The intersection is in a large metropolitan area and an appropriate evaluation time is 60 minutes. The traffic fluctuations determined by arrival flow surveys also indicate a reasonably steady traffic flow during the p.m. peak hour that takes place between 16:30 and 17:30. This time is therefore designated as the evaluation / design period.

The transit peak period, when buses carry the highest number of passengers, takes place between 16:50 and 17:20. The transit assessment time is therefore 30 minutes.

Arrival flows and pedestrian flows

Arrival flows and pedestrian flows per hour during the evaluation/design period were determined by traffic surveys as illustrated in Figure 6.1.2.

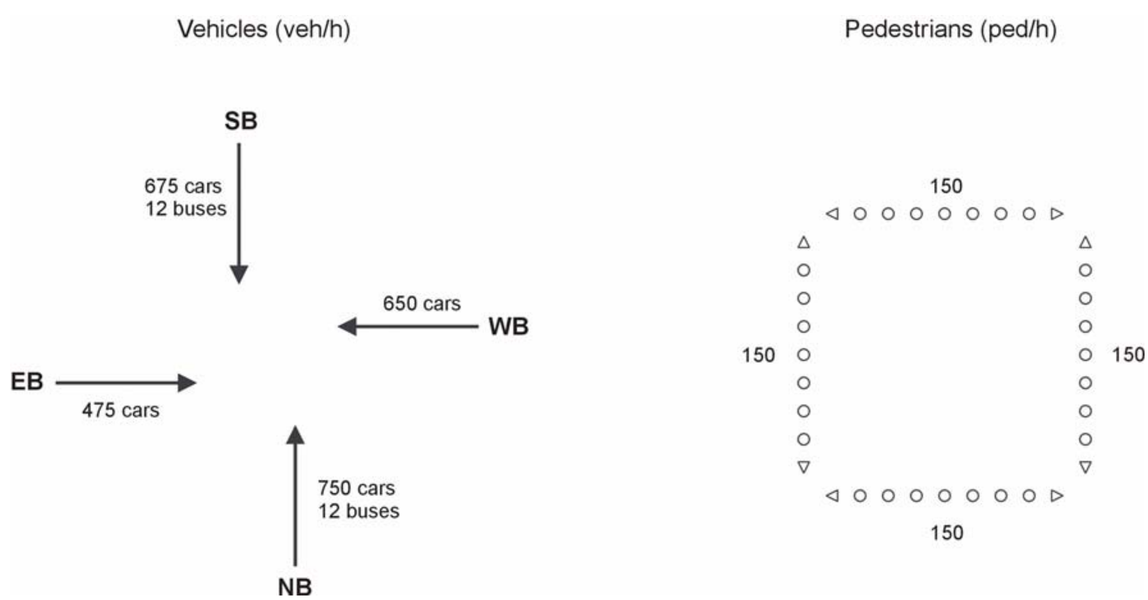


Figure 6.1.2 Vehicular arrival flows and pedestrian flows at the example intersection.

No turning movements are allowed. No adjustments of the arrival flow for the effect of departures during the intergreen period or right turns on red are therefore needed.

Arrival flows on all approaches consisted of passenger cars and buses only. The bus passenger car unit equivalent is 2.0. See Table 3.2 Passenger car unit equivalents on page 3-15.

The average passenger car occupancy is 1.5 persons/car. Buses in the northbound direction carry on average 20 persons and in the southbound direction 10 persons on average. The arrival flows in person/h are calculated for each lane as the products of the number of cars and buses and their occupancies. See 3.1.3 Person flow and vehicle occupancy on page 3-16 and Table 6.1.1.

Table 6.1.1 Summary of arrival flows

Approach lane	Flow of passenger cars per hour	Flow of buses per hour	Arrival flow (veh/h)	Arrival flow (pcu/h)	Arrival flow (person/h)
NB	750 (Given)	12 (Given)	762[1]	774[2]	1365[3]
SB	675	12	687	699	1133
EB	475	0	475	475	713
WB	650	0	650	650	975

1. 750 + 12
2. 750 + (2 x 12)
3. (750 x 1.5) + (12 x 20)

6.1.3 Saturation flows (See 3.2 Saturation Flow on page 3-23)

In this metropolitan area, the basic saturation flow in passenger cars per hour of green for the type of road approaches leading to our intersection has been determined to be 1820 pcu/h. The design conditions assume good pavement surface and good weather.

All approach lanes are 4.0 m wide and have a negligible approach grade. No saturation flow adjustments for the lane width and the grade are therefore necessary. Since no turning movements take place, no adjustments for the radius, left turns, right turns, pedestrians, or shared lanes are applicable. Furthermore, no adjustments are required for transit stops, parking, or queueing or discharge space.

The saturation flow in veh/h will be needed later for several evaluation criteria (Section 4.6). Since only buses are included, it is calculated for all lanes as follows:

$$S_{veh/h} = S_{pcu/h} / S\ (\%q_k\ f_k / 100)$$

$$S_{veh/h} = S_{pcu/h} / (\%\ q_{car}\ 1.0 / 100 + \%\ q_{bus}\ 2.0 / 100)$$

where:

$\%q_k$ =% vehicles of category k in the vehicular arrival flow, and

f_k = passenger car unit equivalent for vehicles category k.

Only the NB and SB approaches are affected because the EB and WB flows consist of passenger cars only. The resulting values of lane saturation flows are summarized in Table 6.1.2:

Table 6.1.2 Determination of adjusted saturation flows

Approach lane	Basic saturation flow (pcu/h)	Adjusted lane saturation flow (pcu/h)	% cars	% buses	Adjusted saturation flow S(veh/h)
NB	1820 (Given)	1820 (Given)	98.43[1]	1.57[2]	1792[3]
SB	1820	1820	98.25	1.75	1789
EB	1820	1820	100	0	1820
WB	1820	1820	100	0	1820

1. 750/762 x 100%
2. 12/762 x 100%
3. 1820 / [98.43 x (1.0/100) + 1.57 x (2.0/100)]

Tentative cycle structure

An examination of the arrival flows and allowable movements indicates that, in this case, the only logical cycle structure is a two-phase operation, as shown in Figure 6.1.3.

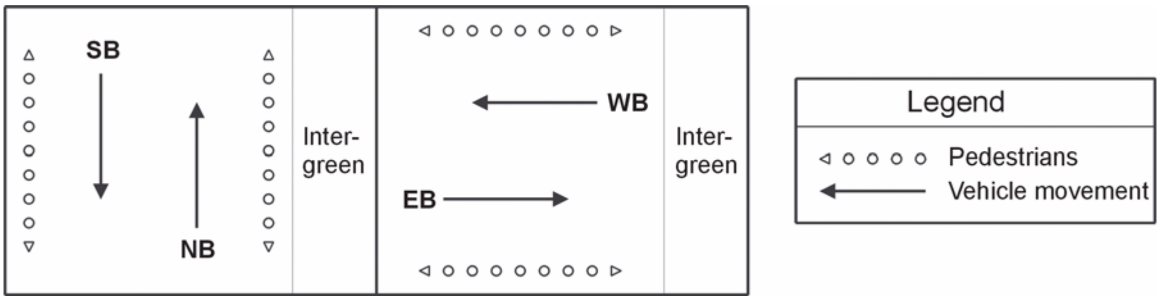


Figure 6.1.3 Cycle structure for the example.

Intergreen periods (See 3.3.3 Intergreen period on page 3-60)

The intergreen period consists of the amber interval and the all-red period. Both are selected or calculated on the basis of the established and accepted local practice. For demonstration purposes, it is assumed that the regional authority over this intersection follows the procedures described in Section 3.3.3 .

Amber interval (See Amber interval on page 3-60)

The standard practice of the traffic administration in the region where this intersection is located calls for 3.0 s amber interval for approaches with a 50 km/h speed limit. Therefore:

$A = 3.0$ s

All-red period (See All-red period on page 3-62)

The all-red period is calculated from the equation:

$r_{all} = I - A$ and

$I = i + (W_c + L_{veh}) / v_c$

where:

i = amber overrun = A - 1 = 3.0 - 1.0 = 2.0 s

Wc = distance to clear = 1.0 +3.0 +4.0 + 4.0 + 3.0 = 15 m

L_{veh} = length of a passenger car = 6.0 m

v_c = clearing speed based on regional practice = 36 km/h = 10.0 m/s.

The intergreen period is then:

$I = 2.0 + (15.0 + 6.0) / 10.0 = 4.1$ s

Standard practice in this region calls for the intergreen period to be rounded to the nearest 1.0 s. Therefore:

$I = 4.0$ s and

$r_{all} = I - A = 4.0 - 3.0 = 1.0$ s for both phases and all approaches.

Lost time (See 3.3.4 Lost time on page 3-64)

Phase lost time is the time following a phase that is not effectively used by traffic. Section 3.3.4 defines it as:

$l_j = I_j - 1.0$

where:

l_j = lost time associated with phase j (s)

I_j = intergreen period between phases j and (j+1) (s).

In this example, all intergreen periods have the same duration. Therefore:

$l_j = I_j - 1.0 = 4.0 - 1.0 = 3.0$ s for all approaches.

6.1.4 Summary of basic vehicular timing requirements

The vehicular timing requirements that constrain the signal timing design are listed in Table 6.1.3.

Table 6.1.3 Basic vehicular timing requirements

Approach lane	Phase	Amber interval (s)	All-red period (s)	Intergreen period (s)	Lost time (s)
NB	1	3.0	1.0	4.0	3.0
SB	1	3.0	1.0	4.0	3.0
EB	2	3.0	1.0	4.0	3.0
WB	2	3.0	1.0	4.0	3.0

Intersection lost time is the sum of the lost time between phases 1 and 2 and between phases 2 and 1. Therefore:

$L = l_1 + l_2 = 3.0 + 3.0 = 6.0$ s.

6.1.5 Pedestrian requirements (See 3.4.1 Pedestrians on page 3-67)

Similarly to the requirements of the intergreen period, it is assumed that the regional authority over this intersection follows the procedures described in Section 3.4.1 .

Pedestrian walk intervals (See Pedestrian walk interval on page 3-67)

There is no pedestrian refuge on any of the crosswalks. Therefore, the minimum pedestrian walk interval is selected:

w_{min} = 10.0 s for all crosswalks.

Pedestrian clearance period (See Pedestrian clearance period on page 3-69)

The pedestrian clearance period is calculated from the formula:

$w_{clear\ i} = W_{ped} / v_{ped}$

where:

W_{ped} = length of the crosswalk measured midway between lines

$W_{ped} = 4.4 + 4.4 = 8.8$ m

v_{ped} = pedestrian walking speed = 1.2 m/s

The resulting value is then:

$w_{clear\ i} = 8.8 / 1.2 = 7.3$ s (rounded up) = 8.0 s for all crosswalks.

Summary of basic pedestrian timing requirements

The pedestrian timing requirements that constrain the signal timing design are listed in Table 6.1.4.

Table 6.1.4 Basic pedestrian timing requirements

Crosswalk	Phase	Walk interval (s)	Clearance period (s)
W	1	10.0	8.0
E	1	10.0	8.0
N	2	10.0	8.0
S	2	10.0	8.0

6.1.6 Signal timing design

Allocation of arrival flows to phases (See 4.2.1 Allocation of arrival flows to phases on page 4-83.)

In this simple case, each approach has only one lane and is controlled by one phase only. Therefore, the arrival flows in each phase are all equal to the full approach arrival flows.

Flow ratios and critical lanes (See 4.2.2 Flow ratio on page 4-85.)

The flow ratios for all lanes are best determined in a tabular format similar to Table 4.1 on page 4-86. The critical lanes are identified and the intersection flow ratio is calculated in Table 6.1.5.

Table 6.1.5 Determination of flow ratios

Lane	Direction	Phase	Adjusted lane arrival flow q (pcu/h)	Adjusted lane saturation flow S (pcu/h)	Lane flow ratio $y = q/S$	Flow ratios for critical lanes y_{crit}
1	NB	1	774 (Table 6.1.1)	1820	0.425[1]	0.425
2	SB	1	699	1820	0.384	
3	EB	2	475	1820	0.261	
4	WB	2	650	1820	0.357	0.357
					Intersection flow ratio $Y = \Sigma y_{crit}$ =	0.782

1. 774 / 1820

The lane numbering system in Column 1 is arbitrary. Some analysts prefer grouping by phases, others like to keep the numbering system independent of the cycle structure.

In Column 2, the designation of the approaches relative to compass directions and direction of travel assist the analyst in orientation. NB represents “northbound”, SB “southbound”, etc. Designations, such as “the north approach” (N), may lead to a confusion of the direction of travel and the location of the approach and should therefore be avoided.

The critical lanes are identified by the highest flow ratios in each phase.

Cycle time determination

(See 4.2.3 Cycle time on page 4-86)

The selection of an appropriate cycle time is guided by the calculated values:

- minimum cycle time,
- optimum cycle time, and
- minimum cycle time needed to accommodate pedestrians.

Additional considerations involve the practical maximum cycle time and system requirements, such as the need for a common cycle time on routes with fixed-time signal coordination.

Minimum cycle time for vehicular flows (See page 4-86)

The minimum cycle time is the shortest cycle time capable of accommodating the arrival flows. It is calculated as:

$c_{min} = L / (1\text{-}Y) = 6 / (1 - 0.782) = 27.52$ s rounds to 28.0

Optimum cycle time for vehicular flows (See page 4-86)

$c_{opt} = (1.5\ L + 5) / (1\text{-}Y) = (1.5 \times 6 + 5) / (1 - 0.782) = 64.22$ s rounds to 64.0 s

Maximum cycle time (See page 4-87)

As stated in Maximum cycle time on page 4-87, the maximum practical cycle time is 120 s. The calculated optimum does not exceed this value.

Minimum cycle time for pedestrians (See page 4-88)

The cycle time for pedestrians must not be shorter than the sum of the longest pedestrian walk intervals and clearance periods for all phases:

$$c_{ped\ min} = \Sigma_j \max (w_{min\ i} + w_{clear\ i})_j$$

where:

$c_{ped\ min}$ = minimum cycle time required for pedestrians (s)

$w_{min\ i}$ = pedestrian walk interval for crosswalk i (See Pedestrian walk interval on page 3-67) (s)

$w_{clear\ i}$ = pedestrian clearance period for crosswalk i (See Pedestrian clearance period on page 3-69) (s)

$\max (w_{min\ i} + w_{clear\ i})_j$ = maximum of the sum of the walk interval plus the corresponding clearance period in each phase (s).

In this case:

$c_{ped\ min} = (10.0 + 8.0) + (10.0 + 8.0) = 36.0$ s.

Cycle time selection (See page 4-88)

Cycle time selection must consider all values calculated above as well as some additional requirements, such as the need for a common cycle time in coordinated systems. In our example, the minimum vehicular cycle time cannot be used, because it cannot accommodate the minimum pedestrian requirements. The minimum pedestrian cycle time cannot also be used because it is too short to be practical. The cycle time should therefore be selected close to its optimum value.

For the purpose of this simple example, the selected cycle time is:

$c = 70.0$ s.

Green allocation (See 4.2.4 Green allocation on page 4-88)

The allocation of green intervals, that is the duration of individual phases, within the cycle time normally employs the proportioning of the total available green time based on the relative values of the critical lane flow ratios for each phase. These ratios reflect the time requirements at full saturation.

Vehicular requirements (See Defining green intervals by balancing flow ratios on page 4-89)

The total available green time within the cycle is:

$\Sigma\, g_j = c - \Sigma\, I_j = 70.0 - (4.0 + 4.0) = 62.0$ s.

where:

$\Sigma\, g_j$ = total green time available in the cycle (s)

c = selected cycle time (s)

I_j = intergreen period following phase j (s).

This total available green time is allocated in proportion to the flow ratio of the critical lane for the corresponding phase and the intersection flow ratio.

For phase 1:

$g_1 = \Sigma\, g_j\, y_1 / Y = 62 \times 0.425 / 0.782 = 33.70$ rounds to 34.0 s

where:

g_1 = green interval for phase 1 (s)

y_1 = flow ratio for the critical lane in phase 1 (Lane flow ratio on page 4-85).

$\Sigma\, g_j$ = total green time available in the cycle (s)

Y = intersection flow ratio (Intersection flow ratio on page 4-85).

Similarly, for phase 2:

$g_2 = \Sigma\, g_j\, y_2 / Y = 62 \times 0.357 / 0.782 = 28.30$ rounds to 28.0 s

The resulting values represent the actual green intervals, not the effective green intervals. It is useful to verify that the sum of the rounded longest green intervals for each phase and the intergreen periods equals the selected cycle time, as indicated in Table 6.1.6.

Table 6.1.6 Summary of vehicular timing

Interval or period	Notation	Duration(s)
Green, phase 1	g_1	34
Intergreen, phase 1	I_1	4
Green, phase 2	g_2	28
Intergreen, phase 2	g_2	4
Cycle time	c	70

This signal timing has been determined on the basis of vehicular flows and vehicular intergreens only. The duration of individual phases must then be tested whether it can accommodate minimum pedestrian requirements.

Pedestrian phase requirements (See page 4-89)

Each phase must be able to accommodate at least the minimum walk interval and the associated pedestrian clearance period. Therefore, we must test whether the following condition is met:

$(g_j + I_j) \geq \max\,(w_{min\ i} + w_{clear\ i})_j$

where:

g_j = green interval for phase j (s)

I_j = intergreen period following phase j (s)

$w_{min\ i}$= walk interval for crosswalk i (s)

$w_{clear\ i}$ = pedestrian clearance period for crosswalk i (s)

$\max\,(w_{min\ i} + w_{clear\ i})_j$ = maximum of the sum of the walk interval and pedestrian clearance period (s).

In our case:

Phase 1:$(34 + 4) \geq (10 + 8)$

Phase 2:$(28 + 4) \geq (10 + 8)$

The minimum pedestrian requirements are met for both phases.

Summary of the designed signal timing

The overview of the designed signal timing is illustrated in the timing diagram illustrated in Figure 6.1.4.

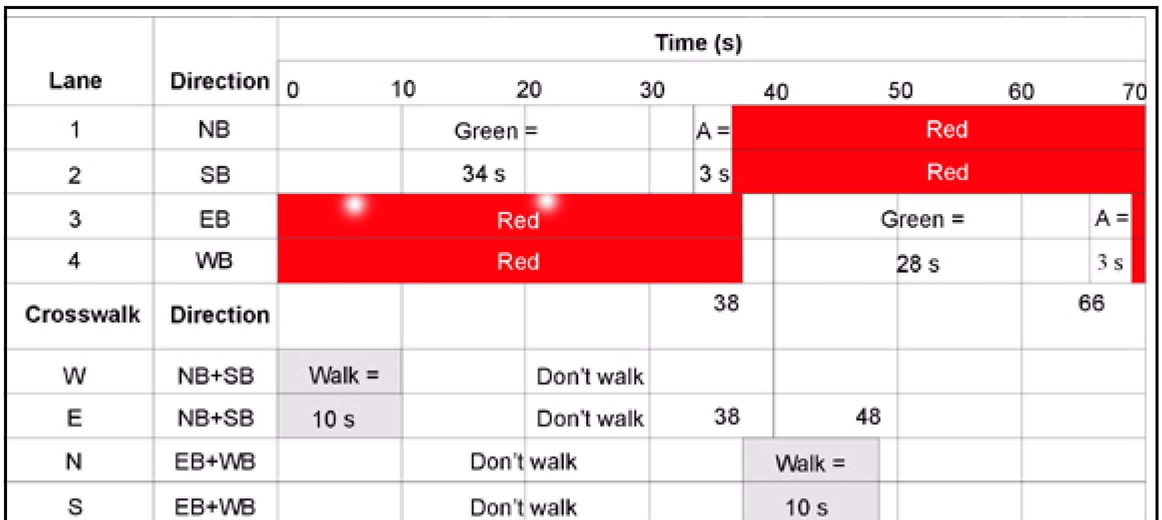


Figure 6.1.4 Final signal timing design shown in a timing diagram. Note: Part of the pedestrian clearance period may be displayed as a flashing Don't Walk Interval

This signal timing must be evaluated to determine the quality of service provided for the drivers, pedestrians and passengers, as well as some of the impacts on the adjacent area. Moreover, the restricted distance between the stop line and the exit from a shopping centre requires assessment of the queue reach.

6.1.7 Evaluation

In this example, most major criteria from all three evaluation categories will be applied. They include criteria related to capacity, criteria related to queueing and other operational and environmental criteria. This example may also serve as a demonstration of an operational analysis, keeping in mind that in such a detailed analysis many input parameters, such as lane arrival flows and lane saturation flows, may have been observed directly. Moreover, some of the evaluation criteria, such as the various forms of delay, overflow factor or the reach of queues may also be measured. The comparison of such measured and calculated values allows a local calibration of the procedures described in Chapter 3:"Analysis" on page 3-13 and Chapter 4:"Planning and Design" on page 4-81. Appropriate survey methods are described in Chapter 5:"Surveys" on page 5-123.

Criteria related to capacity (see 4.7 Evaluation criteria related to capacity on page 4-96)

Evaluation criteria relevant in this example include:

- capacity
- degree of saturation, and
- probability of discharge overload.

All of these measures are determined for individual lanes.

Capacity and degree of saturation (See 4.7.1 Capacity of approach lanes for vehicular traffic on page 4-96, and 4.7.2 Degree of saturation on page 4-97)

The calculation is shown in a tabular format in Table 6.1.7.

The effective green interval applied is $g_e = g + 1.0$.

The relatively high degrees of saturation, that is the flow-to-capacity ratios, in both critical lanes 1 and 4 indicate that some operational difficulties may be expected. It is therefore advisable to evaluate the performance further using other measures of effectiveness.

Table 6.1.7 Determination of capacity and degree of saturation

Lane	Direction	Phase	Effective green interval g_e (s)	g_e/c	Adjusted lane arrival flow q (pcu/h)	Adjusted lane saturation flow S (pcu/h)	Lane capacity $C=S\ g_e/c$ (pcu/h)	Degree of saturation $x = q/C$
1	NB	1	35	0.500[1]	774 (Table 6.1.1)	1820	910[2]	0.851[3]
2	SB	1	35	0.500	699	1820	910	0.768
3	EB	2	29	0.414	475	1820	754	0.630
4	WB	2	29	0.414	650	1820	754	0.862

1. 35 / 70
2. 1820 x 0.500
3. 774 / 910

Probability of discharge overload (see 4.7.3 Probability of discharge overload and overload factor on page 4-97)

The calculation of the probability of discharge overload or its determination from the graph in Figure 4.3 on page 4-98 requires the average number of arrivals per cycle and the average cycle capacity as inputs. These values are derived from the arrival flows and from the previously computed hourly capacities by dividing them by the number of cycles per hour.

$$P_{discharge\ overload} = 1 - [P(X \le X_c)]^2$$

where:

$P(X \le X_c)$ = arrival overload probability, i.e., the probability that the number of arrivals in a cycle (X) will be equal or less than cycle capacity X_c:

$$P(X \le X_c) = S\ mi\ [e^{(-m)}] / i!$$

where:

$P(X \le X_c)$ = probability of the number of arrivals in a cycle (X) being less than or equal to cycle capacity (X_c), with

$X_c = C/n$ = cycle capacity (pcu/cycle)

C = capacity (pcu/h)

n = number of cycles in an hour = 3600 / c

c = cycle time (s)

S = summation for i = 0, 1, 2, 3,..., X_c

m = mean of the arrival distribution with the cycle time as the counting interval, or:

m = q / n = q c / 3600 (pcu/cycle), with

q = arrival flow (pcu/h)

The calculation is shown in Table 6.1.8.

Table 6.1.8 Determination of probability of discharge overflow

Lane	Direction	Phase	Average number of arrivals per cycle m=q/n (pcu)	Average capacity per cycle X_c=C/n (pcu)	Probability of discharge overload $P_{disch\ overl}$
1	NB	1	15.1[1]	17.7[2]	0.362[3]
2	SB	1	13.6	17.7	0.200
3	EB	2	9.2	14.7	0.064
4	WB	2	12.6	14.7	0.401

1. 774 / (3600/70)
2. 910 / (3600/70)
3. $P(X \le X_c) = P(X \le 17.7) = \Sigma m^i [\exp(-m)] / i! = \Sigma 15^i [\exp(-15)] / i!$
for all i = 0, 1, 2,....17, 17.7. The value for i = 17.7 was obtained by interpolation of probabilities based on i = 17 and i = 18. If a rounded integer value of 18 were applied, the resulting probability of discharge overload for lane 1 would be 0.328, indicating the same problem magnitude. A similar value can be obtained by interpolation from the graph in Figure 4.3 on page 4-98.

The resulting probabilities of discharge overload suggest that, in the critical lanes 1 and 4, about 35% to 40% of the cycles may be overloaded. That means that many drivers would have to wait for more than 1 cycle. The problem definition states that the intersection is isolated. No signal coordination is therefore considered. In a hypothetical case that traffic progression were desired with the upstream intersection located south of this intersection, it would be difficult to achieve because more than one third of the cycles feature overflows. In that case, it would be advisable to redesign the intersection and to reduce the probability of discharge overload before proceeding with the design of signal coordination.

Evaluation criteria related to queueing (See 4.8 Criteria related to queueing on page 4-101)

The criteria investigated in this example include:

- average overall lane delay in s/pcu;
- average overall intersection delay in s/pcu;
- average overall lane delay in s/veh;
- total person delay;
- average delay to pedestrians;
- number of stops;
- queues at the end of the red interval;
- average queue length; and
- maximum probable queue length.

Average overall lane delay in s/pcu (See page 4-101)

The designer would be interested both in the delays that vehicles encounter in individual lanes and the average overall intersection delay as an overall indicator of the quality of service provided by the designed signal timing.

The basic equation for estimating the average overall delay is as follows:

$$d = k_f\, d_1 + d_2$$

where:

d = average overall delay (s/pcu)

k_f = adjustment factor for the effect of the quality of progression, with

$k_f = (1 - q_{gr}/q)\, f_p \,/\, (1-g_e/c)$ (calculated values are also given in Figure 4.3 on page 4-98), and

q_{gr}/q = proportion of vehicles arriving during the green interval

f_p = supplemental adjustment factor for platoon arrival time

d_1 = average overall uniform delay (s/pcu)

d_2 = average overflow delay (s/pcu), with

$d_1 = c\,(1 - g_e/c)^2 \,/\, [2\,(1 - x_1\, g_e/c)]$, and

$$d_2 = \left[(x-1) + \sqrt{(x-1)^2 + (240x)/(Ct_e)}\right] 15_{te}$$

where:

c = cycle time (s)

g_e = effective green interval (s) (Green interval and effective green interval on page 3-58)

x_1 = minimum of (1.0, x)

x = degree of saturation (See Section 4.7.2 on page 4-97.)

q = arrival flow (pcu/h)

C = capacity (pcu/h) (See Section 4.7.1 on page 4-96.)

t_e = evaluation time in minutes (See Section 3.1.5 on page 3-17).

Since this intersection is isolated and therefore the arrivals are random, the arrival type is AT3 and the value of both the progression adjustment factor k_f and platoon arrival adjustment factor f_p are 1.0 (Table 4.5 on page 4-103).

The design cycle time is:

$c = 70$ s,

the evaluation time determined during the analysis stage is

$t_e = 1$ hour $= 60$ minutes,

and the adjustment factor for the effect of the quality of progression for random arrival pattern from Table 4.5 on page 4-103 is

$k_f = 1.0.$

Since all other variables in the equations for the average overall delay components are related to individual lanes, the input values and the calculated results are shown in Table 6.1.9.

Table 6.1.9 Determination of the average overall delay for passenger car units

Lane	Direction	Phase	g_e/c	Lane capacity C (pcu/h)	Degree of saturation x=q/C	Average overall uniform delay d_1 (s/pcu)	Average overall overflow delay d_2 (s/pcu)	Average overall delay d (s/pcu)
1	NB	1	0.500	910	0.0.851 (Table 6.1.7)	15.22[1]	10.82[2]	26.05[3]
2	SB	1	0.500	910	0.768	14.21	6.45	20.66
3	EB	2	0.414	754	0.630	16.25	4.04	20.29
4	WB	2	0.414	754	0.862	18.68	14.12	32.80

1. $70\left(1-0.500\right)^2 \div \left[2\left(1-\frac{774}{910}(0.500)\right)\right]$

2. $15 \times 60\left[\left(\frac{774}{910}-1\right)+\sqrt{\left(\left(\frac{774}{910}\right)-1\right)^2+\frac{240\left(\frac{774}{910}\right)}{910(60)}}\right]$

3. $1.0d_1 + d_2$

While the magnitudes of the average overall uniform delay are similar for all lanes, the average overall overflow delay for lane 1, and especially lane 4, are significantly higher than for lanes 2 and 4. This indicates a potential problem during some portions of the evaluation period when the arrival flows are higher than the average.

The criteria for deleting the overflow delay component would apply only for lanes 2 and 3, with their saturation flow values over 1000 pcu/h and degrees of saturation less than 0.8. The order of the average overall delay magnitude would remain within the same range and within the accuracy of the formula for these relatively short delays.

Average overall intersection delay

The average overall intersection delay is calculated as the weighted average of the average overall lane delays for all intersection lanes:

$$d_{int} = \Sigma_j \Sigma_i q_{ij} d_{ij} / \Sigma_j \Sigma_i q_{ij}$$

where:

q_{ij} = arrival flow in lane i in phase j (pcu/h)

d_{ij} = average overall delay for vehicles in lane i departing in phase j (s/pcu)

$\Sigma_j \Sigma_i$ = summation over individual lanes i and over phase j.

The necessary input is organized in Table 6.1.10 which yields the total overall intersection delay. Dividing it by the sum of all arrival flows gives the average overall intersection delay.

Table 6.1.10 Determination of the total overall intersection delay

Lane	Direction	Phase	Arrival flow q_{ij} (pcu/h)	Average overall delay d_{ij} (s/pcu)	Weighted delay $\Sigma_j \Sigma_i q_{ij} d_{ij}$ (s/h)
1	NB	1	774	26.05	20160
2	SB	1	699	20.66	14441
3	EB	2	475	20.29	9637
4	WB	2	650	32.80	21317
			Σ = 2598		Σ = 65555

Therefore:

$$d_{int} = \Sigma_j \Sigma_i q_{ij} d_{ij} / \Sigma_j \Sigma_i q_{ij} = 65555 / 2598 = 25.23 \text{ s/pcu}$$

This value is consistent with the previously determined capacity related measures. The red intervals are 33 s for the north-and southbound approaches, and 39 s for the east-and westbound approaches. The average overall intersection delay is somewhat longer than one half of the red intervals. This was to be expected given the higher probabilities of discharge overload for the critical lanes. Judging only by this criterion, the intersection operation appears to be acceptable.

Average overall delay in veh/h (See page 4-105)

Since vehicles other than passenger cars (buses in lanes 1 and 2) constitute less than 85% of the arrival flows, the difference between this type of delay and the average overall delay expressed in pcu/h is small. The calculated average overall delay in pcu/h may therefore be used. Nevertheless, to demonstrate the process and the nature of the difference, the average overall delay in veh/h are calculated in Table 6.1.11.

Table 6.1.11 Determination of the average overall delay for vehicles

Lane	Direction	Arrival flow from (Table 6.1.1) q (veh/h)	Saturation flow from (Table 6.1.2) S (veh/h)	Average overall uniform delay d_1 (s/veh)	Average overall overflow delay d_2 (s/veh)	Average overall delay d (s/veh)
1	NB	774	1792	15.22	10.82	26.05
2	SB	699	1789	14.21	6.45	20.66
3	EB	475	1820	16.25	4.04	20.29
4	WB	650	1820	18.68	14.12	32.80

In this example, working with vehicles rather than in passenger car units yields no practical difference in the values of the average overall delay. Note that the average uniform delay in veh/h is identical to its value in pcu/h.

Total person delay, not including pedestrians (See Non-vehicular delay on page 4-105)

The total overall person delay is calculated as follows:

$$D_{person} = d_{veh}\ \Sigma\ (V_k\ O_k) / 3600$$

where:

D_{person} = total person delay during the transit assessment time in the given lane (h)

d_{veh} = average overall delay per vehicle (Average overall delay in s/veh on page 4-105) based on the transit assessment time (See Section 3.1.5 on page 3-17) (s/veh). Note that, in this case, the transit assessment time applied in the delay formula in Table 6.1.12 is t_e = 30 minutes as identified in Analysis period, evaluation time and transit assessment time on page 6-144.

O_k = average occupancy of vehicles category k during the transit assessment time (person/veh)

V_k = volume of vehicles category k during the transit assessment time (veh/assessment time).

The sum of all these person delays in all lanes that receive the green signal indication in the given phase is then the total person delay during the transit assessment time in that phase:

$$D_{person\ phase\ j} = \Sigma_j\ D_{person\ lane\ ij}$$

and, for the whole intersection, the sum of all person delays during the transit assessment time in all phases is:

$$D_{person\ int} = \Sigma_j\ D_{person\ phase\ j} / 3600$$

where:

$D_{person\ int}$ = total intersection person delay during transit assessment time (h).

Table 6.1.12 Average overall delay per vehicle during transit assessment time

Lane	Direction	Arrival flow q (veh/h)	Average uniform delay d_1 (s/veh)	Average overflow delay d_2 (s/veh)	Average overall delay d (s/veh)
1	NB	762 (Table 6.1.1)	15.22[1]	10.45[2]	25.67[3]
2	SB	687	14.21	6.36	20.57
3	EB	475	16.25	4.02	20.26
4	WB	650	18.68	13.46	32.14

1. $70\left(1-0.500\right)^2 \div \left[2\left(1-\frac{774}{910}(0.500)\right)\right]$
2. $d\left(15 \times 30\left[\left(\frac{774}{910}-1\right)+\sqrt{\left(\left(\frac{774}{910}\right)-1\right)^2+\frac{240\left(\frac{774}{910}\right)}{910(30)}}\right]\right)$
3. $1.0d_1 + d_2$

The average overflow delay has been re-calculated in a similar fashion as illustrated in Table 6.1.9, but with the transit assessment time applied as the evaluation time t_e = 30 min. The difference, however, is small because individual intersection approaches are not oversaturated and the average uniform delay in Table 6.1.12 is independent of the evaluation time. The calculation of the total person delay for each of the two phases is shown in Table 6.1.13.

Table 6.1.13 Determination of total person delay

Lane and direction	Average overall delay in 30 min d (s/veh)	Volume of cars V_{car} (veh/30 min)	Average car occupancy O_{car} (person/veh)	Volume of buses V_{bus} (veh/30 min)	Average bus occupancy O_{bus} (person/veh)	Total person delay for lane D_{person} (s)	Total person delay for phase D_{person} (h)
1 NB	25.67	375	1.5	6	20	17520[1]	
2 SB	20.57	338	1.5	6	10	11660	8.11[2]
3 EB	20.26	238	1.5	0	0	7219	
4 WB	32.14	325	1.5	0	0	15668	6.36

1. (375 x 1.5 + 6 x 20) x 25.67
2. (17520 + 11660) / 3600

During the 30-minute transit assessment time, the total person delay for Phase 1 is 26% greater than the total person delay in Phase 2. Yet, 1250 people travel through Phase 1 and only 845 people in Phase 2; that is, Phase 1 has 48% higher person flow. Such analysis may prompt a re-allocation of green intervals to provide a more equitable distribution of person delays.

Average delay to pedestrians (See page 4-106)

Under the assumption of random pedestrian arrivals at a given crosswalk and its operation below the crosswalk capacity, the average pedestrian delay is independent of the pedestrian flows and calculated as:

$$d_{ped} = (c - w)^2 / 2c$$

where:

d_{ped} = average delay to pedestrians (s)

c = cycle time (s)

w = walk interval (s).

In our case, the cycle time is 70 s, and all walk intervals are 10 s. Therefore:

$$d_{ped} = (c - w)^2 / 2c = (70 - 10)^2 / 2 \times 70 = 25.7 \text{ s} \quad \text{at all crosswalks.}$$

Number of stops (See section 4.8.3 on page 4-107)

The number of vehicles that are stopped at least once by the signal operation during the evaluation time can be derived under the assumption of a random arrival pattern as:

$$N_s = k_f\, [q\, (c - g_e) / 3600\, (1 - y)]\, [60\, t_e / c] = [k_f\, t_e\, q\, (c - g_e)] / [60\, c\, (1 - y)]$$

where:

N_s = number of stopped vehicles during the evaluation time (pcu) $\leq (q\, t_e / 60)$

k_f = adjustment factor for the effect of the quality of progression from Average overall delay in s/pcu on page 4-101

q = arrival flow (pcu/h)

g_e = effective green interval (s)

c = cycle time (s), adjusted for prevailing conditions

y = flow ratio = y = q / S, capped at 0.99, with

q = arrival flow (pcu/h)

S = saturation flow (pcu/h)

t_e = evaluation time (min)

In our example, the cycle time is 70 seconds, the evaluation time is 60 minutes and the progression factor for all approaches is 1.0. The number of stops is calculated in Table 6.1.14.

Table 6.1.14 Determination of number of stops

Lane	Progression factor k_f	Arrival flow q (pcu/h)	Effective green g_e (s)	Flow ratio y	Number of stops N_s
1	1.0	774	35	0.425[1]	673[2]
2	1.0	699	35	0.384	567
3	1.0	475	29	0.261	376
4	1.0	650	29	0.357	592

1. 774 / 1820
2. [1 x 60 x 774 x (70-35)] / [60 x 70 x (1- 0.425)]

A comparison of the arrival flow and the number of stops indicates that most of the drivers must stop either because of the red signal indication or because they must join the stopped end of the queue. In the second case, the front of the queue may already be discharging.

Queues (See Section 4.8.4 on page 4-108)

The following queue information is determined in order to demonstrate the calculation processes:

- queues at the end of the red interval;
- average queue reach; and
- the maximum probable queue reach for all lanes.

Queues at the end of the red interval (See page 4-108)

The average queue at the end of the red interval in undersaturated conditions is:

$$Q_r = q\ (c - g_e) / 3600$$

where:

Q_r = average queue at the end of the red interval (pcu)

q = arrival flow (pcu/h)

c = cycle time = 70 s

g_e = effective green interval (s).

The queue lengths for individual lanes are calculated in Table 6.1.15.

Table 6.1.15 Determination of average queue length at the end of the red interval

Lane	Arrival flow q (pcu/h)	Effective green g_e (s)	Average queue length at the end of red Q_r (pcu)	Average queue length at the end of red Q_r (m)
1	774	35	7.5[1]	45[2]
2	699	35	6.8	41
3	475	29	5.4	32
4	650	29	7.4	44

1. 774 (70 - 35) / 3600
2. 7.5 x 6.0m

The length of the space taken by a passenger car unit to determine the values in Column 5 is assumed to be 6.0 m. The front of the queue will start discharging at the end-of-red point in time but the newly arriving vehicles will be still joining the rear of the queue. The more critical consideration is therefore how far this upstream moving rear of queue will reach before the discharging front of the queue “catches up” with it.

Average queue reach (See page 4-109)

In this case, only the conservative estimate of the average value of the longest distance at which the vehicles join the stopped end of the queue in individual cycles is applied. The reason for this are the relatively high degree of saturation and high probabilities of discharge overload in the critical lanes. The calculation uses the following formula:

$$Q_{reach} = q\ c / 3600$$

where:

q = arrival flow (pcu/h)

c = cycle time = 70 s.

Again, a tabular computation format is adopted, as shown in Table 6.1.16.

Table 6.1.16 Determination of average queue reach

Lane	Arrival flow q (pcu/h)	Average queue reach Q_{reach} (pcu)	Average queue reach Q_{reach} (m)
1	774 (Table 6.1.1)	15.1[1]	91[2]
2	699	13.6	82
3	475	9.2	55
4	650	12.6	76

1. (774 x 70) / 3600
2. 15.1 x 6.0m

Consistent with previous calculations, the length of the space taken by a passenger car unit to determine the values in Table 6.1.16 is assumed to be 6.0 m. As expected, the calculated average queue reach distances are significantly grater than the queues lengths at the end of the red interval.

Maximum probable queue reach (See page 4-110)

This measure takes into account the randomness in the number of vehicles arriving during individual cycles. The probability of a queue reach exceeding a given length (expressed as number of pcu) may be read from the graph in Figure 4.7 on page 4-111 or approximated as:

$$P(Q_i > Q) = 1 - [P(Q_i \le Q)]^2$$

where:

$P(Q_i > Q)$ = probability that a given queue reach Q will be exceeded, and

$P(Q_i \le Q)$ = distribution of queue reaches with the calculated average queue reach Qreach as the mean, and expressed in the cumulative form:

$$P(Q_i \le Q) = \Sigma (Q_{reach})^j [e^{(-Qreach)}] / j!$$

with:

Q_{reach} = average queue reach estimate (pcu) from the appropriate liberal or conservative formula (See Average queue reach on page 4-109)

j = summation parameter, representing queue states 0, 1, 2, 3,...., Q.

In this case, the queue reach is not really critical, with the exception of lane 4 that has a somewhat limited queueing space of 90 m. For all lanes, the maximum probable queue reach will be calculated at the 5% probability level, that is, the calculated queue reach will be exceeded only in 5% of the cycles. That means that in 95% of cycles the required queueing distance will be shorter. A summary of the calculations is shown in Table 6.1.17.

Table 6.1.17 Determination of maximum probable queue reach

Lane	Average queue reach (pcu)	Maximum probable queue reach (pcu at 5%)	Maximum probable queue reach (m at 5%)
1	15.1	23[1]	138[2]
2	13.6	21	126
3	9.2	16	96
4	12.6	20	120

1. Figure 4.7 on page 4-111
2. 23 x 6.0m

It is obvious that, for lane 4, the queues in some cycles will reach beyond the shopping centre exit and may block it. The question is then how often will this happen.

The available queueing distance of 90 m to the shopping centre exit is sufficient for 90m/6m = 15 pcu. Applying the previous summation formula, the probability level can be determined as 36.3%. A similar value may be obtained by interpolation in Figure 4.7 on page 4-111 for the curve Q_{reach} = 12.6 pcu and the farthest probable queue reach of 15.0 pcu on the horizontal axis. Therefore, it may be expected that the shopping centre exit will be blocked during the end of the red interval and during the first part of the green interval in over one third of the cycles. This may represent a substantial portion of the evaluation time which, in this case, coincides with the design hour. This information, together with the assessment of the parking pattern, makes it possible to examine the interaction of the arrival flow and the flows exiting from the shopping centre.

6.2 Worked Example 2: T-Intersection with Turning Movements

6.2.1 Basic Information

The intersection volumes and configurations are shown below:

GIVEN:

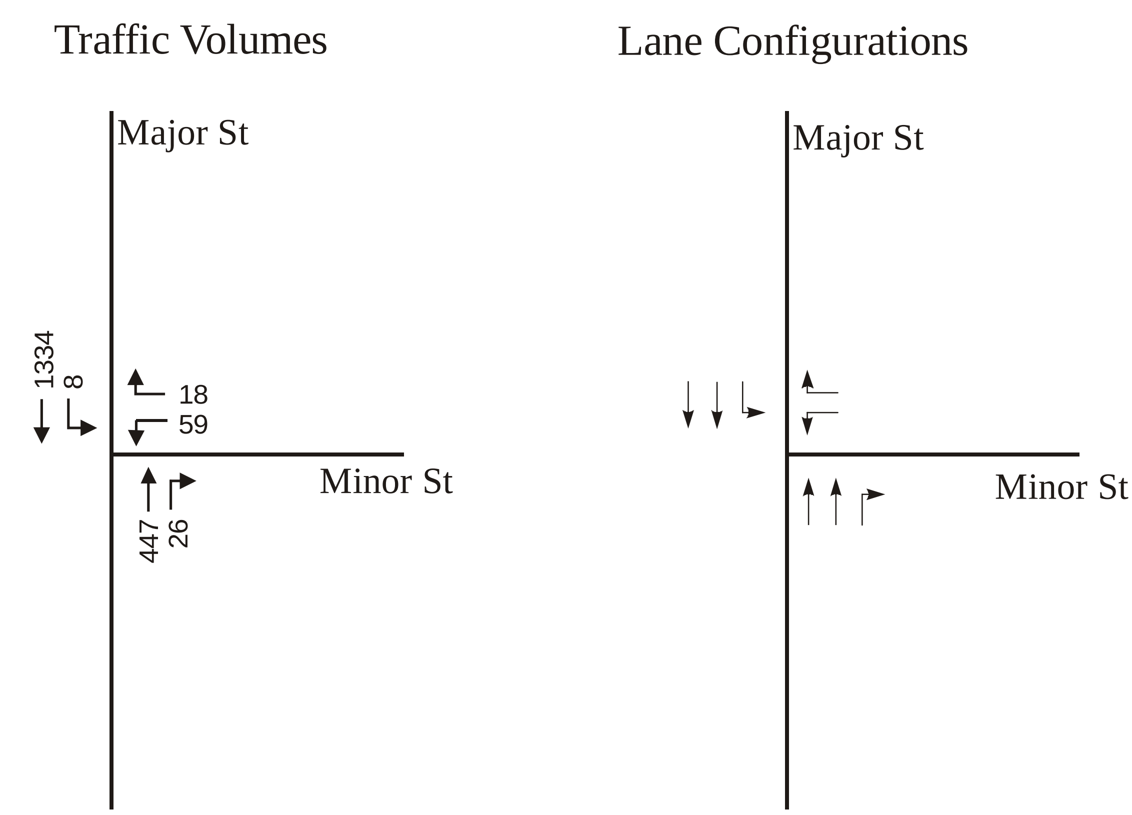
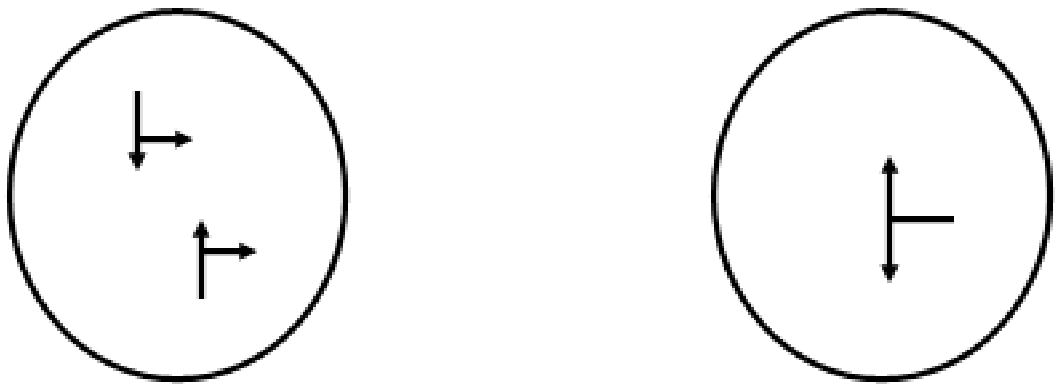


Figure 6.2.1 Intersection layout for the example

Key factors:

- Adjustments: 10% medium-size vehicles
- 1 truck = 2.0 pcu on all approaches.
- LTOI = 2 for southbound
- Protected Left-turn Saturation Adjustment = 1.05
- Progression Factor = 3

Phasing and timings



North-South
Green 90
Amber 4
Red 2

Westbound
Green 18
Amber 4
Red 2

Figure 6.2.2 Phasing and timings for the example

Cycle time is c= 120

Therefore:

Table 6.2.1 Timings for the example

		Green	Amber	All Red	Lost Time	g_e
N/S	Main	90	4	2	5	91
WB	Main	18	4	2	5	19

6.2.2 Adjusted Volume Calculations

All approaches:

q_c = 90% passenger cars

q_t = 10% trucks where 1 truck = 2.0 pcu

$$q = (q_c) + (q_t) \times 2.0$$

Example: SB Through

E_q for SB through = 1334

q=(1334 x 0.9) + (1334 x 0.1) x 2.0

=1200.6 + 266.8

=1467

Summary

Table 6.2.2 Volumes Summary

Approach	Movement	Demand Volume, q (veh/h)	Adjusted Volume q_{adj} (pcu/h)
SB	Th	1334	1467
	Lt	8	9
NB	Rt	26	29
	Th	447	492
WB	Rt	18	20
	Lt	59	65

6.2.3 Saturation Adjustment Calculations

1. Initial Saturation - 1850

2. For Lefts

a. No adjustment for geometric or traffic conditions required

b. Adjust for permissive flow (no peds opposing)

Calculate opposing flow: **Example: SB LT** (See Table 3.15 on page 3-43)

i.) $q'_o = q_o {}^* c/g_{ei}$

$q'_o = 492^*\ 120/91$

$q'_o = 649$

ii.) $F_L = 1.05e^{(-0.00121fq'_o)} - 0.05$

f is from Table 3.16 on page 3-43

f = 0.625 for 2 opposing lanes

$\therefore F_L = 1.05e^{(-0.00121*0.625*649)} - 0.05$

$F_L = 0.593$

iii.) $S_{adj} = S_{Basic} {}^* F$

$S_{adj} = 1850\ {}^*0.593$

$S_{adj} = 1097$

3. For right turns:

No saturation adjustments required

$\therefore$ S = 1850

6.2.4 Summary Calculation for WB Left

Volume — Demand volume = 59
Adjusted volume = 65 (from above)
Clearing volume = 65

Timing — green = 18
amber = 4
all red = 2
lost = 5
g_e = 18+4+2-5 = 19

Saturation — WB LT is unopposed and therefore "protected" (See: Protected left turns in exclusive lane on page 3-40.)
Thus the LT protected factor applies

$S_{adj} = 1.05 \text{ x } S_{basic}$

$S_{adj} = 1.05 \text{ x} 1850$

$S_{adj} = 1943$

y Ratio

$$y_i = \frac{q_{iadj}}{s_{iadj}}$$ (See: Lane flow ratio on page 4-85)

$$y_i = \frac{65}{1943}$$

$y_i = 0.033$

Capacity

$$C = S_{adj} \times \frac{g_e}{c}$$

$$C = 1943 \times \frac{19}{120}$$

$C = 308$ but also need to add LTOI

V/C ratio (lane degree of saturation) (See: Section 4.7.2 Degree of saturation on page 4-97)

$$V/C = \frac{V_{adj}}{C}$$

$$V/C = \frac{65}{308}$$

$V/C = 0.211 = x$

Delay Calculations (See: Section 4.8.1 Vehicle delay on page 4-101)

Uniform delay:

$d_1 = c(1\text{-}g_e/c)^2 / [2(1\text{- x } g_e/c)]$

$d_1 = 120(1\text{-}19/120)^2 / [2(1\text{-}0.21 \text{ x } (19/120))]$

$d_1 = 120(0.842)^2 / 2(0.967)$

$d_1 = 44.0\text{s}$

Overflow delay:

$$d_2 = 15t_e[(x-1) + \sqrt{(x-1)^2 + 240 * \frac{x}{(C \times t_e)}}]$$

$$d_2 = 15 \times 60[(0.21-1) + \sqrt{(0.21-1)^2 + 240 * \frac{0.21}{(308 \times 60)}}]$$

$d_2 = 1.1\text{s}$

Total delay:

$d = k_f d_1 + d_2$

$k_f = 1$ for progression factor = 3

$d = 1 \text{ x } 44.0 + 1.6$

$d = 45.6\text{s}$

Queue Calculations (See: Section 4.8.4 Vehicular queues on page 4-108)

Average Q:

$Q = (q \text{ x } c)/3600$

$Q = (65 \text{ x } 120)/3600$

$Q = 2.2$ *vehs*

6.2.5 Summary

Table 6.2.3 Summary of the Example

Movement		# of lanes	Phase	q veh/h	Q_{adj} pcu/h	RTOI/ LTOI	q e/ea pcu/h	S_{adj} pcu/h	Total S_{adj} pcu/h	y	Critical y	g_e sec	Total C pcu/h	x	d_1 sec	d_2 sec	d sec	Total Q_{cons} pcu	Q_{cons} pcu/ lane
SB	**T**	2	1	1334	1467	0	146	1850	3700	0.396	0.396	91	2806	0.523	5.8	0.7	6.5	48.9	24.5
	L	1	1	8	9	9	0	1097	1097	0.000		91	892	0.011	3.5	0.0	3.5	0.3	0.3
NB	**R**	1	1	26	29	0	29	1850	1850	0.016		91	1403	0.021	3.6	0.0	3.6	1.0	1.0
	T	2	1	447	492	0	492	1850	3700	0.133		91	2806	0.175	4.0	0.2	4.2	16.4	8.2
WB	**R**	1	2	18	20	0	20	1850	1850	0.011		19	293	0.068	43.0	0.5	43.5	0.7	0.7
	L	1	2	59	65	0	65	1943	1943	0.033	0.033	19	308	0.211	44.0	1.6	45.6	2.2	2.2
										Overall	0.429								

Overall Level of Service (See: Section 4.7.4 Level of Service on page 4-99)

$$\frac{V}{C_{overall}} = \Sigma\frac{q_\phi}{S_\phi} \times \frac{c}{g_e}$$

where $\Sigma\frac{q_\phi}{S_\phi}$ = the sum of the critical q/S values for each phase

$$\frac{V}{C_{overall}} = 0.429 \times (120/110)$$

= 0.468

LOS = A

6.3 Worked Example 3: Four-legged Intersection

6.3.1 Basic information

Figure 6.3.1 illustrates an isolated signalized intersection located in Toronto. Lanes are 3.5m wide and all curbs have a radius of 16m. Approach grades are all less than 2%. There is no transit. The intersection is to be evaluated during the peak period of a typical weekday. Traffic counts conducted between 7 and 8 a.m. are included in the volume data provided in Table 6.3.1. Pedestrian flow rates are provided in Figure 6.3.2. The posted speed limit is 50 km/hr on the north/south street and 60 km/hr on the east/west street. Basic saturation flow in the area is 1700 pcu/h of green time. The Peak Hour Factor is 0.92. It can be assumed for this intersection that all left and right turn lanes have sufficient storage. Use an evaluation time period of 15 minutes.

The existing signal timing plan consists of 4 phases:

- Phase 1: 6 seconds advanced green interval serving northbound traffic followed by 3 seconds amber and 1 second all red interval.
- Phase 2: 22 seconds green interval serving northbound and southbound traffic followed by 3 seconds amber and 2 seconds all red interval.
- Phase 3: 6 seconds east/west left turning green arrow followed by 3 seconds east west left turning amber and 1 second all red.
- Phase 4: 28 seconds green interval serving eastbound and westbound traffic followed by 3 seconds amber and 2 seconds all red interval.

Total cycle length is 80 seconds.

Assume:

1 left turn on intergreen per cycle for shared straight and left turning lanes

2 left turns on intergreen per cycle for exclusive left turn lanes

2 right turns on red per cycle for exclusive right turning lanes

Arrival type on all directions follows a progression type 3.

Table 6.3.1 Traffic volumes for the example

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Cars/hour	164	417	36	107	491	63	198	628	109	38	342	41
Heavy vehicle percentage	1	1	1	0.5	0.5	0.5	2	2	2	1.5	1.5	1.5

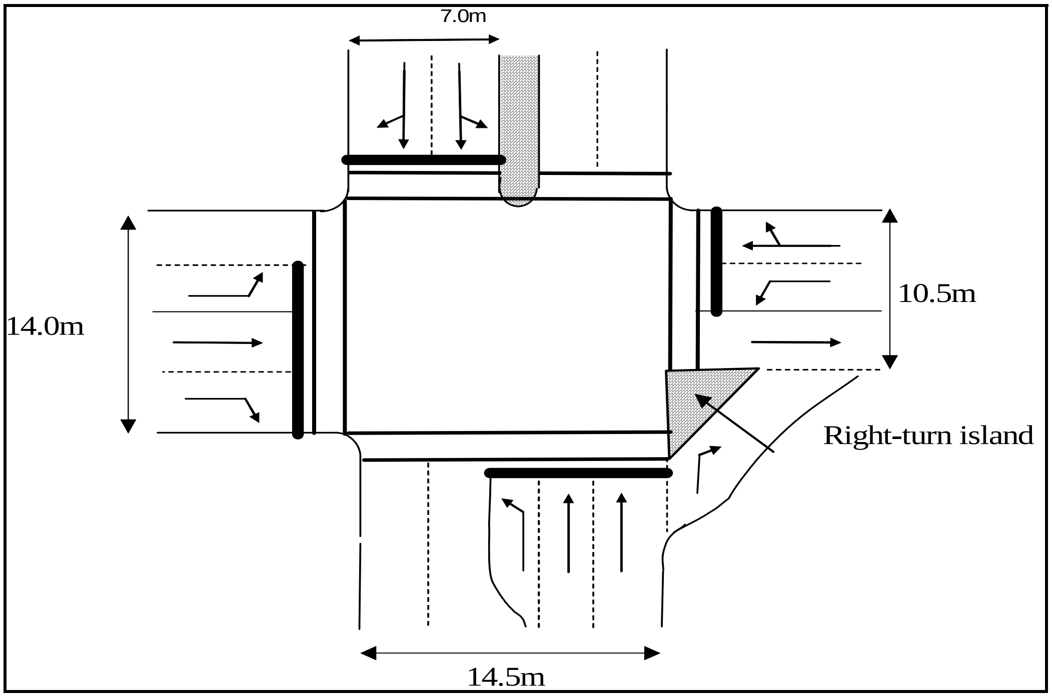


Figure 6.3.1 Intersection Geometry

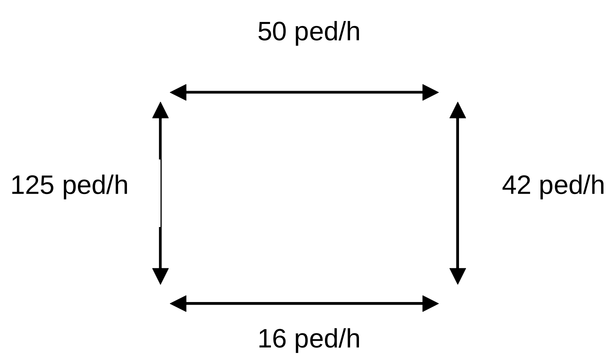


Figure 6.3.2 Observed Pedestrian Flows

6.3.2 Solution Approach

1. Convert all movement volumes to pcu/h.
2. Determine saturation flow rate adjustment factors for each lane.
3. Determine the adjusted saturation flow rate for each lane for each phase during which the lane discharges.
4. Determine capacity for each lane.
5. Allocate flow to lanes for movements having more than one lane available for use.
6. Determine delay for each lane.

6.3.3 Part 1: Convert flows to pcu/h

Use formula:

$$q_{adj} = \sum_k f_k q_k$$ See page 3-15

Eastbound (Sample for Table 6.3.2)

PCU equivalents are shown in Table 3.2 on page 3-15.

Left turn: Cars to PCU: 164 x 0.99 x 1.0 = 162.36

Heavy Vehicles to PCU: 164 x 0.01 x 2.0 = 3.28

Total: 162.36 + 3.28 =165.64

Adjustment for PHF (See Peak Hour Factor on page 3-16)

$$V_{adj} = \frac{165.4}{0.92} = 180$$

Table 6.3.2 Adjusted volumes

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Flow (pcu/h)	180	458	40	117	536	69	220	696	121	42	377	45

6.3.4 Part 2: Determination of Saturation Flow Rate Adjustment Factors

(See from Section 3.2.7 on page 3-39 to Section 3.2.13 on page 3-52)

Westbound Movements

For westbound left-turn, shared right and through lanes:

$q_r = 69$, $q_t = 536$, $q_l = 117$ *pcu/h*

since grade is less than 2%, no geometric adjustments are required.

a. Westbound right turn movement

$q_{ped} = 50$/hr, north crosswalk

$q'_{ped} = q_{ped} \times c/g = 50$ ped/h $\times$ 80/28 = 143 ped/h (See Table 3.18 on page 3-47)

Since $q'_{ped} < 200$ ped/h, impact of pedestrians can be ignored and $F_{Rped} = 1.0$

Since $S_R = 1700 \text{ x } 1.0 = 1700$ pcu/h (See page 3-31)

$K_R = S_T/S_R = 1700/1700 = 1.0$ (See Table 3.19 on page 3-49)

Therefore S_{RT} can be treated same as $S_T = 1700$.

b. Westbound left turn movement

Westbound left turn lane movement performance is calculated in two steps:
Protected in Phase 1,
Permissive in Phase 2.
(i) protected (No peds, no geometric restrictions)
For a protected left turn movement, $F_L = 1.05$ (See page 3-41)
Therefore,

$$S_{LADJ} = S_{L_{BASIC}} \times F_L$$ (See page 3-31)

$$S_{LADJ} = 1700 \times 1.05 = 1785$$

(ii) permissive phase (Peds and opposing vehicles)

Factoring opposing vehicles

Opposing flow consists of straight through volume only. (See page 3-42)

$q'_o = q_o \times c/g_{eo}$

$q'_o = 458 \text{ x } 80/29$

$q'_o = 1263$ pcu

Since there is one lane of opposing through traffic, opposing flow lane factor $f = 1.0$ (See Table 3.16 on page 3-43)

$f \text{ x } q'_o = 1263$ pcu

Factoring pedestrians (See Table 3.18 on page 3-47)

Crossing the south approach = 16 ped/h

$q'_{ped} = q_{ped} \times c/g$ = 16 ped/h × 80/28 = 46 ped/h

which is considered to be minimal and will therefore not affect the left-turn flow.(See page 3-43)

Factoring opposing vehicles

$$F_L = 1.05e^{-0.00121 f q'_o} - 0.05$$

$$F_L = 1.05e^{-0.00121(1263)} - 0.05$$

$$F_L = 0.1777$$

$$S_{Ladj} = 1700 \text{ x } 0.17764 = 302 \text{ pcu/h}$$

LTOI

Number of signal cycles in one hour = 3600/80= 45 cycles.

Total LTOI = 45 x 2 = 90 pcu

Therefore the capacity of exclusive left-turn lane

C = capacity during protected phase + capacity during permissive phase + capacity during intergreen phase. (See page 3-20)

C = 1785 x 7/80 + 302 x 29/80 + 2 LTOI x 3600/80

C = 355.66 = 356pcu/h

While calculating the measures of effectiveness, it should be noted that the left turns on intergreen are removed from the calculations.

Hence westbound left turn clearing volume = 117 - 90 = 27 pcu

Capacity during Phase 1 = 1785 x 8/80 = 156 pcu

Hence degree of saturation during Phase 1 = 27/156 = 0.15

Since all WB-L vehicles clear during Phase 1, Phase 2 degree of saturation = 0

c. Westbound shared right and through movements

It has been determined earlier that S_{RT} can be treated same as S_T.

Therefore total volume on the shared through and right turning lane

q = (536 + 69) pcu

q = 605 pcu

Capacity (C)

= 1700 x 29/80

= 616 pcu/h

Degree of saturation

= 605/616 = 0.982

Eastbound Movements

a. Eastbound right turn movement

Pedestrian flows on the south side = 16 / hr

$q'_{ped} = q_{ped} \times c/g$ = 16 ped/h × 80/28 = 46 ped/h (See Table 3.18 on page 3-47)

Ped/h < 200, therefore F_{Rped} = 1.00

Right-turn capacity (assuming 2 pcu/cycle for RTOR)

C = capacity during green phase + capacity during red phase

C = (1700 x 29/80) + (2 x 3600/80) = 616 + 90 = 706 pcu

For calculating the degree of saturation, remove the right turn on red volumes

= 45 x 2 = 90 pcu

There are only 40 right turning pcu. So all the pcu can get cleared during the red phase.

Therefore degree of saturation = 0

b. Eastbound through movement

Saturation flow = 1700 pcu/h

$C_{ij} = S_{ij}\, g_{ej}/c$ (See page 4-96)

C = 1700 x 29/80

C = 616 pcu/h

Degree of saturation

= 458/616 = 0.74

c. Eastbound left turn movement

Pedestrian factor:

Crossing north approach volume = 50/hr

$$q'_{ped} = q_{ped} \times c/g = 50 \text{ ped/h} \times 80/28 = 143 \text{ ped/h}$$

Assuming 80 sec cycle length, there will be 45 cycles/hr. This means there will be approximately 143/45 = 3 pedestrians per cycle waiting to cross the road. This volume is considered too small to be included in the left turn adjustment factor. (See page 3-43)

Opposing westbound vehicle factor:

$$q'_o = q_o \times c/g_e$$

$$q'_o = (536+69) \text{ x } 80/29 = 1669 \text{ pcu}$$

(Note: because the westbound approach configuration includes a shared through and right turning lane, the westbound right turn volume has been included as part of the opposing volume)

Saturation flow factor for the left turn during the permissive phase: (See Table 3.15 on page 3-43)

$$F_L = 1.05e^{-0.00121fq'_o} - 0.05$$

$$F_L = 1.05e^{-0.00121(1.0 \text{ x } 1669)} - 0.05$$

$$F_L = 0.089$$

Saturation flow rate during the protected phase:

$F_L = 1.05$ (See page 3-41)

Left turn capacity = capacity for protected phase + capacity for permissive phase + capacity for LTOI

$$C = (1.05 \text{ x } 1700 \text{ x } 7/80) + (1700 \text{ x } 0.089 \text{ x } 29/80) + (3600/80 \text{ x } 2)$$

$$C = 156.2 + 54.8 + 90 = 301 \text{ pcu}$$

Calculating the measures of effectiveness:

- Left turns on intergreen are removed from the calculations.
- Number of signal cycles in one hour = 3600/80= 45 cycles.
- Total LTOI = 45 x 2 = 90 pcu
- Hence eastbound left turn clearing volumes = 180 - 90 = 90 pcu
- Eastbound left turning capacity during Phase 3 = (1700 x 1.05) 7/80 = 156 pcu
- Hence all remaining 90 vehicles clear during Phase 3 with a degree of saturation = 90/156 = 0.58

Therefore there are no remaining eastbound left turning pcu during Phase 4. Hence degree of saturation for this movement during Phase 4= 0

Northbound Movements

a. Northbound right turn movement

This is not a right turn channelized movement. The lane configuration is best treated as an exclusive right turning lane since the movement does not have its own discharge lane or upstream storage. A shorter length right turn storage lane could be treated as a shared right through lane with an allowance of a few vehicles during the 'red' period. A true channelized movement includes long upstream storage and a separate discharge or merge lane downstream. A true channelized movement can be assumed to operate as a 'free' movement for which capacity is not constrained by the intersection.

Pedestrian flows on the east approach = 42/hr

= 42 x 80/32 = 105 ped/h

Ped/h < 200, therefore $F_{Rped} = 1.0$

Capacity is defined by the effective green capacity + the RTOI capacity:

$C\text{-}S_{BASIC} \times F_{ADJ} \times g_e/c + (X_{RTOI} \times n)$ (See page 3-21 and page 3-31)

C= 1700 x 1.0 x (7+23+3)/80 + (2 x 3600/80)

= 701.3 + 90

= 791 pcu

Clearing right turning volume = 121 – 90 = 31 pcu

Capacity during Phase 1 = 1700 x 7/80 = 148.75 = 149 pcu

Degree of Saturation = 31/149 = 0.21

b. Northbound Through Movement

There are 2 lanes, each with a saturation flow of 1700 pcu/h

C = 2 x 1700 x ((7+23+3)/80)

C = 1403 pcu/h

Capacity during Phase 1 = 2 x 1700 x 7/80 = 297.5 = 298 pcu

Total clearing vehicles = 696 pcu

Assuming the movement operates at capacity during Phase 1 (i.e. 298 pcu), the degree of saturation during Phase 1 = 298/298 = 1

During Phase 2, clearing volume = 696 - 298 = 398 pcu

Capacity during Phase 2 = 2 x 1700 x (33 - 7)/80 = 1105 pcu

Degree of Saturation during Phase 2 = 398/1105 = 0.36

c. Northbound left turn movement

There are two phases:

(i) protected,

(ii) permissive.

(i) Protected (No peds, no geometric restrictions)

$$S_{Ladj} = S_{L_{Basic}} \times F_L$$

$$S_{Ladj} = 1700 \times 1.05 = 1785$$

(ii) Permissive phase (Peds and opposing vehicles)

Factoring opposing vehicles (See Table 3.15 on page 3-43)

$$q'_o = q_o \times c/g_e$$

(q_o does not include the left turn flow in a shared left turn and straight-through lane)

$$q'_o = (377+45) \text{ x } 80/23 = 1468 \text{ pcu}$$

Factoring opposing pedestrians

(i.e. those crossing the west approach)

$$q'_{ped} = q_{ped} \times c/g = 125 \text{ ped/h} \times 80/22 = 455 \text{ ped/h}$$

$$= 10 \text{ peds/cycle}$$

The pedestrian volume can be considered minimal in terms of having any effect on the saturation flow. (See page 3-43)

$F_L = 1.05e^{-0.00121 f q'_o} - 0.05$ $f = 0.625$ for two lanes (See Table 3.16 on page 3-43)

$$F_L = 1.05e^{-0.00121(0.625 \text{ x } 1468)} - 0.05$$

$$F_L = 0.296$$

Therefore saturation flow for left turns during permissive phase

$$S_{Perm} = 1700 \text{ x } 0.296$$

$$S_{Perm} = 503 \text{ pcu}$$

Capacity for left turns = (i) capacity during protected phase + (ii) capacity during permissive phase + (iii) capacity during intergreen phase

$$C = (1785 \text{ x } 7/80) + (503 \text{ x } 23/80) + (2 \text{ x } 3600/80)$$

$$C = 156.2 + 144.6 + 90$$

$$C = 391 \text{ pcu/h}$$

Calculating the degree of saturation:

- Remove LTOI = 90 pcu
- Therefore clearing volume = 220 - 90 = 130 pcu
- Capacity during the protected phase = 156 pcu

Therefore Degree of Saturation in Phase 1 = 130/156 = 0.83

Southbound Movements

$q_L = 42$

$q_T = 377$

$q_R = 45$

Follow steps in Figure 3.2 on page 3-19 to determine equivalent flows per lane

Determine S_L (As if in exclusive left lane)

Opposing flow in advanced phase

Note that the northbound movements are spread across two phases (1 and 2) while the southbound left turns are permitted only during Phase 2. The portion of northbound traffic proceeding during Phase 1 must therefore be excluded in order to determine the opposing flow for the southbound left turns.

Opposing flow = (2 x 1700) x 7/80 = 298 pcu

$q_o = 696 - 298 = 398$ pcu

$q'_o = q_o \times C/g_e = 398 \text{ x } 80/23 = 1384.35$ pcu

$F_L = 1.05e^{-0.00121x0.625x1384.35}-0.05$

$F_L = 0.319$

$S_L = 1700$ pcu/h x 0.319 = 542 pcu/h

Pedestrians on east approach = 42 ped/h

This equates to less than 1 ped per cycle which is negligible (See page 3-45)

Determine S_R (As if in exclusive lane)

Pedestrians crossing the west leg = 125 ped/h

$q'_{ped} = q_{ped} \text{ x } C/g_e = 125 \text{ x } 80/22 = 455$ pcu

Since q'_{ped} is greater than 200, calculate the adjustment factor from Table 3.18 for Toronto

$F_{RPed} = 0.60 - (q'_{ped} / 8516) = 0.6 - 455/8516 = 0.55$

$S_R = 1700 \text{ x } 0.55 = 935$ pcu/h

$S_T = 1700$ pcu/h

Calculate turning factors (Step a Figure 3.2)

$K_L = S_T/S_L = 1700/542 = 3.14$

$K_R = S_T/S_R = 1700/935 = 1.8$

Calculate equivalent through flows (Step b Figure 3.2)

$q'_L = q_L K_L = 0 \text{ X } 3.14 = 0$ pcu

$q'_T = q_T \text{ x } 1.0 = 377$ pcu

$q'_R = q_R K_R = 45 \text{ x } 1.8 = 81$ pcu

Calculate the average equivalent through flows per lane (Step c Figure 3.2)

$q_1 = q'_L + q'_T + q'_R = 0 + 377 + 81 = 458$ pcu

Number of lanes = 2

Equivalent through flow per lane

$q'_{Lane} = 458/2 = 229$ pcu

(Step d Figure 3.2)

$q'_{TL} = q'_{Lane} - q'_L = 229 - 0 = 229$ pcu

$q'_{TR} = q'_{Lane} - q'_R = 229 - 81 = 148$ pcu

(Step e Figure 3.2)

$q_{TL} = q'_L / K_L + q'_{TL} = 0/3.14 + 229 = 229$ pcu

$q_{TR} = q'_R / K_R + q'_{TR} = 81/1.8 + 148 = 193$ pcu

Shared through/right lane (Table 3.19)

$q'_T = q'_{Lane} = 229$ pcu

$F_{TR} = (q_R + q_T)/ q'_T = (45+148)/229 = 0.84$

$S_{TR} = 1700 \text{ x } 0.84 = 1428$ pcu/h

$C_{TR} = 1428$ pcu/h x $23/80 = 410$ pcu/h

V/C or x = $(45+148)/410 = 0.47$

Shared through/left Lane (Table 3.17)

$q'_T = q'_{Lane} = 229$ pcu

$F_{TL} = (q_L + q_T)/ q'_T = (0 + 229)/229 = 1.00$

$S_{LT} = 1700 \text{ x } 1.00 = 1700$ pcu/h

$C_{TL} = 1700$ pcu/h x $23/80 + 1 \text{ x } 3600/80 = 534$ pcu/h

V/C or x = $(42 + 229)/534= 0.51$

6.3.5 Delay Calculations

WB-Left

$g_e = (6+1) + (28+1) = 36$ sec

$g_e/c = 36/80 = 0.45$

Arrival type = AT3

$k_f = 1$ (See Table 4.5 on page 4-103)

$f_p = 1$

t_e = Evaluation interval = 15 mins.

$x = q_i / C_i = 117 / 367 = 0.329$ (See page 4-97)

$d_1 = c\,(1 - g_e/c)^2/[2(1\text{-}x_1 g_e/c)]$

$d_1 = 80\,(1\text{-}0.45)^2 / [2(1\text{-}0.329 \times 0.45)]$

$d_1 = 14.2$ sec/pcu

$d_2 = 15_{te}[(x-1) + \sqrt{(x-1)^2 + (240x)/(Ct_e)}]$ (See page 4-100)

$d_2 = 15 \times 15[(0.329 - 1) + \sqrt{(0.329 - 1)^2 + (240\text{x}0.329)/(356 \times 15)}]$

$d_2 = 2.459 = 2.5$ sec

Average overall delay

$d_o = k_f d_1 + d_2$ (See page 4-101)

$d_o = 1 \times 14.2 + 2.5$

$d_o = 16.7$ sec/pcu

$d_o = 17$ sec/pcu

SB-Through/Right:

$x_1 = 193/410 = 0.47$

$g_e/c = 23/80 = 0.2875$

SB-Through/Left:

$x_1 = 271/534 = 0.51$

SB-Through/Right:

$d_1 = 80\,(1\text{-}0.2875)^2 / 2(1\text{-}0.47\text{x}0.2875)$

$d_1 = 23.48$ sec/pcu

$d_2 = 15 \times 15[(0.47 - 1) + \sqrt{(0.47 - 1)^2 + (240 \times 0.47)/(410 \times 15)}$

$d_2 = 3.84$ sec/pcu

$d_0 = 27$ sec/pcu

SB-Through/Left:

$d_1 = 80\ (1\text{-}0.2875)^2 / [2(1\text{-}0.51\text{x}0.2875)]$

$d_1 = 23.78$ sec/pcu

$$d_2 = 15 \times 15[(0.51 - 1) + \sqrt{(0.51 - 1)^2 + (240 \times 0.51)/(534) \times 15)}]$$

$d_2 = 3.45$ sec/pcu

$d_0 = 27$ sec/pcu

WB-Through/Right:

$g_e = 28 + 1 = 29$ sec

$g_e/c = 29/80 = 0.36$

Arrival type = AT3

$k_f = 1$ (See Table 4.5 on page 4-103)

$f_p = 1$

t_e = Evaluation interval = 15 mins.

$x_1 = q_1 / C_1 = 605/616 = 0.982$

$d_1 = c\ (1 - g_e/c)^2/[2(1\text{-}x_1 g_e/c)]$

$d_1 = 80\ (1\text{-}0.36)^2 / [2(1\text{-}0.982\text{x}0.36)]$

$d_1 = 25.24$ sec/pcu

$$d_2 = 15t_e[(x - 1) + \sqrt{(x - 1)^2 + (240x)/(Ct_e)}]$$

$$d_2 = 15 \times 15[(0.982 - 1) + \sqrt{(0.982 - 1)^2 + (240\text{x}0.982)/(616 \times 15)}]$$

$d_2 = 32.14$ sec/pcu

Average overall delay = $k_f d_1 + d_2$

= 25.24 + 32.14

= 57.4 sec/pcu

= 57 sec/pcu

Arrival type = AT3

$k_f = 1$ (See Table 4.5 on page 4-103)

$f_p = 1$

t_e = Evaluation interval = 15 mins.

R: $x_1 = q_1/C_1 = 121 / 791 = 0.153$

T: $x_1 = q_1/C_1 = 348 / 701 = 0.496$

L: $x_1 = q_1/C_1 = 220 / 391 = 0.563$

NB-Right:

$d_1 = c\,(1-g_e/c)^2 / [2(1-x_1 g_e/c)]$

$d_1 = 80\,(1-0.4125)^2 / [2(1-0.153 \text{ x } 0.4125)]$

$d_1 = 14.74$ sec/pcu

$$d_2 = 15t_e[(x-1) + \sqrt{(x-1)^2 + (240x)/(Ct_e)}]$$

$$d_2 = 15 \times 15[(0.153-1) + \sqrt{(0.153-1)^2 + (240\text{x}0.153)/(791 \times 15)}]$$

$d_2 = 0.41$ sec/pcu

$d_0 = 15$ sec/pcu

NB-Through:

$d_1 = 80\,(1-0.4125)^2 / [2(1-0.496 \text{ x } 0.4125)]$

$d_1 = 17.36$ sec/pcu

$$d_2 = 15 \times 15[(0.496-1) + \sqrt{(0.496-1)^2 + (240\text{x}0.496)/(701 \times 15)}]$$

$d_2 = 2.50$ sec/pcu

$d_0 = 20$ sec/pcu

NB-Left:

$d_1 = 80\,(1-0.375)^2 / [2(1-0.563 \text{ x } 0.375)]$

$d_1 = 19.80$ sec/pcu

$$d_2 = 15 \times 15[(0.563-1) + \sqrt{(0.563-1)^2 + (240\text{x}0.563)/(391 \times 15)}]$$

$d_2 = 5.75$ sec/pcu

$d_0 = 26$ sec/pcu

EB:

R: $x_1 = q_1/C_1 = 40/706 = 0.057$

T: $x_1 = q_1/C_1 = 458/616 = 0.74$

L: $x_1 = q_1/C_1 = 180/301 = 0.598$

EB-Right:

$d_1 = 80\ (1\text{-}0.36)^2\ /\ 2(1\text{-}0.057 \text{ x } 0.36)$

$d_1 = 16.60$ sec/pcu

$$d_2 = 15 \times 15[(0.057 - 1) + \sqrt{(0.057 - 1)^2 + (240\text{x}0.057)/(706 \times 15)}]$$

$d_2 = 0.15$ sec/pcu

$d_0 = 17$ sec/pcu

EB-Through:

$d_1 = 80\ (1\text{-}0.36)^2\ /\ [2(1\text{-}\ 0.74 \text{ x}0.36)]$

$d_1 = 22.25$ sec/pcu

$$d_2 = 15 \times 15[(0.74 - 1) + \sqrt{(0.74 - 1)^2 + (240\text{x}0.74)/(616 \times 15)}]$$

$d_2 = 7.93$ sec/pcu

$d_0 = 30$ sec/pcu

EB-Left:

$d_1 = 80\ (1\text{-}0.45)^2\ /\ [2(1\text{-}0.598 \text{ x } 0.45)]$

$d_1 = 16.56$ sec/pcu

$$d_2 = 15 \times 15[(0.598 - 1) + \sqrt{(0.598 - 1)^2 + (240\text{x}0.598)/(301 \times 15)}]$$

$d_2 = 8.50$ sec/pcu

$d_0 = 25$ sec/pcu

6.3.6 Summary

Table 6.3.3 Summary of the example

Movement		Number of lanes	Phase	q_{adj} pcu/h	RTOI/LTOI	q_{adj} pcu/h	Total S_{adj} pcu/h	Total C pcu/h	x (V/C)	y	Critical y
NB	R	1	1	121	90	31	1700	149	0.21	0.018	
	T	2	1	298	0	298	3400	298	1.00	0.088	0.088
	L	1	1	220	90	130	1785	156	0.83	0.073	
SB	TR	1	2	193	0	193	1426	410	0.47	0.135	0.135
	TL	1	2	271	42	229	1700	534	0.47	0.135	
NB	R	1	2	0	0	0	1700	642	0.00	0.00	
	T	2	2	398	0	398	3400	1105	0.36	0.117	
	L	1	2	0	0	0	503	235	0.00	0.00	
WB	L	1	3	117	90	27	1785	156	0.17	0.015	
EB	L	1	3	180	90	90	1785	156	0.58	0.050	0.05
WB	TR	1	4	605	0	605	1700	616	0.98	0.356	0.356
	L	1	4	0	0	0	302	199	0.00	0.00	
EB	R	1	4	40	40	0	1700	706	0.00	0.00	
	T	1	4	458	0	458	1700	616	0.74	0.269	
	L	1	4	0	0	0	152	145	0.00	0.00	
										Overall	0.629

Overall Level of Service (See: Section 4.7.4 Level of Service on page 4-99)

$$\frac{V}{C_{overall}} = \Sigma \frac{q_\phi}{S_\phi} \times \frac{c}{g_e}$$

where $\Sigma \frac{q_\phi}{S_\phi}$ = the sum of the critical q/S values for each phase

$$\frac{V}{C_{overall}} = 0.629 \times (80/66) = 0.76$$

LOS = C

6.4 Worked Example 4: Four-legged Intersection in Edmonton

6.4.1 Basic Information and Calculations

- S_{basic} = 1700 pcu/hg
- g_e = g + 1
- PHF = 0.92
- Heavy vehicles = 2.0 pcu
- All lanes 3.6m wide
- Approach grades:

  North leg +1.35 % (SB approach)

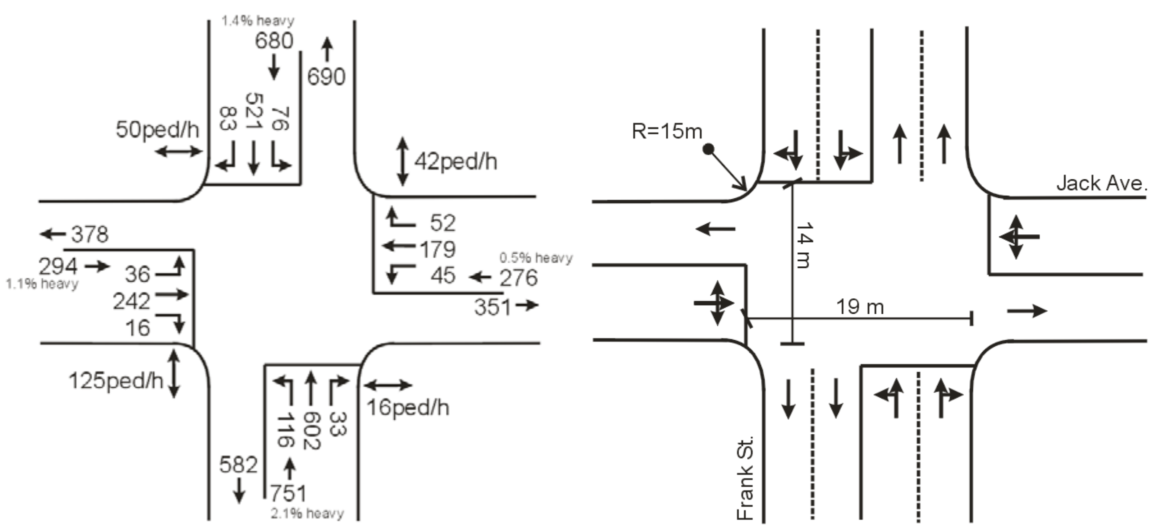
  South leg -1.20 % (NB approach)

  West leg +3.05 % (EB approach)

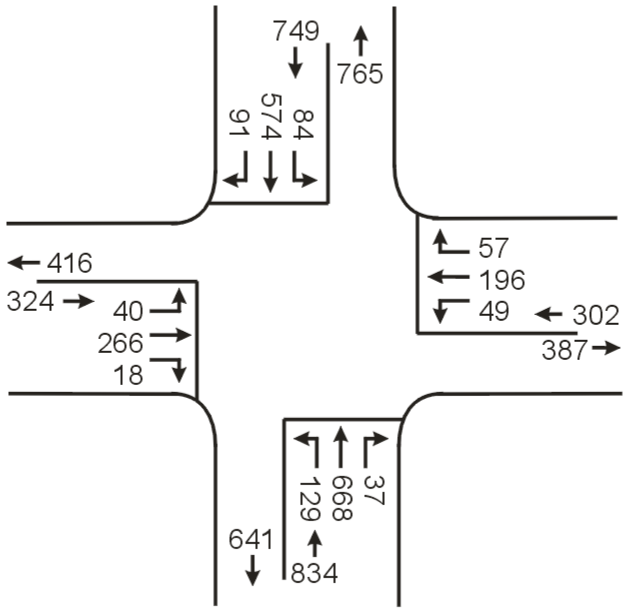
  East leg +1.85 % (WB approach)
- All RT radii =15 m
- No transit
- Speed limit:N/S street = 40 km/h; E/W street = 40 km/h
- 1 LTOI for any shared LT/Through lane
- 2 LTOI for exclusive LT lane
- 0 RTOR for any shared RT/Through lane
- 1 RTOR for exclusive RT lane
- N/S arrival type = AT3
- E/W arrival type = AT2
- te = 15 minutes for delay calculations

Figure 6.4.1 Intersection Turning Movement Diagram and Intersection Geometry Lane Designations

AM Peak Hour Volumes



Convert TMC to PCU's peak flow rate



Sample calculation:

$$EBT_{HEAVY} = 242 \times 1.1\% \times = 2.0 = 5.3$$

$$EBT_{CAR} = 242 \times 98.9\% \times = 1.0 = 293.3$$

$$5.3pcu + 293.3pcu = 245pcu$$

$$q_{ADJPHF} = \frac{245pcu}{0.92} = 266$$

Signal Timing Plan (Simple 2-Phase Operation)

- N/S permissive 32s ped walk = 15s
- N/S amber 3s ped clear = 20s
- N/S all red 1s
- E/W permissive 20s ped walk = 10s
- E/W amber 3s ped clear = 13s
- E/W all red 1s
- Cycle time = 60s
- N/S g_e = 32 + 1 = 33s
- E/W g_e = 20 + 1 = 21s

6.4.2 Saturation Flow Rate Adjustments

Westbound shared LTR lane

$q_R = 57$ pcu/h $\qquad q_T = 196$ pcu/h $\qquad q_L = 49$ pcu/h

Given the left and right turn volumes are of similar magnitude, test which combination (through-right or through-left) is more critical. (See page 3-51)

Westbound through/right:

$$q'_{ped} = q_{ped} \times c/g_e = 50 \times \frac{60}{20} = 150$$

$$\text{Since } 150\text{ped} < 200\text{ped} \rightarrow F_{RPED} = 1.0$$

$$\therefore S_{RT} = S_T = 1700\frac{\text{pcu}}{\text{h}}$$

Westbound through/left:

$$q'_{opp} = q_{opp} \times c/g_e = (266 + 18 + 40) \times \frac{60}{21} = 925.7\text{pcu}$$

$$F_L = 1.05e^{-0.00121fq'_o} - 0.05 = 1.05e^{-0.00121(1.0)(925.7)} - 0.05 = 0.293$$

There are only 16 pedestrians per hour on the south leg. This equates to less than one pedestrian on every other cycle. Therefore the impact is considered to be negligible. (See page 3-44)

$$S_{LT} = 1700 \times 0.293 = 497.3\text{pcu/h}$$

$$K_L = \frac{S_T}{S_L}\text{(Table 3.17)} = \frac{1700}{497.3} = 3.418$$

Calculate the total equivalent flow (Table 3.17):

$$q'_T = K_L q_L + q_T = 3.418(49) + 196 + 57 = 420.5\text{pcu}$$

$$F_{TL} = \frac{(q_L + q_T)}{q'_T} = \frac{(49 + 196 + 57)}{420.5} = 0.718$$

$$S_{LTR} = 1700 \times 0.72 = 1220.6\text{pcu/h}$$

$$\text{Capacity} = 1220.6\text{pcu/h} \times \frac{21}{60} = 427.1\text{pcu/h}$$

$$V/C = \frac{57 + 196 + 49}{427.1} = 0.707$$

Eastbound LTR lane

q_R = 18 pcu/h q_T = 266 pcu/h q_L = 40 pcu/h

G = +3.05% this is greater than 2.0% so an adjustment must be determined

$$F_{GRADE} = 1 - (G + HV) = 1 - (0.0305 + 0.011) = 0.9585$$

$$S_{ADJ} = (1700) \times (0.9585) = 1629.5 pcu/h$$

(See page 3-31)

This is the new S_{BASIC} for EB.

Since this is a shared LTR lane, the through-left will be the critical movement since the volume is higher than the right turn and pedestrian volumes are not large.

LT

$$q_{opp} = WB + WBRT \text{ in one lane}$$

$$q^j{}_{opp} = 196 + 57 + 49 = 302 pcu/h$$

$$q_{opp} = q_{opp} \times c/g_e = 302 \times 60/21 = 862.9 pcu/h$$

Pedestrian volumes are low enough that they can be ignored for the LT (See page 3-44).

F=1.0 (Table 3.16)

$$F_L = 105e^{(-0.00121)(862.9)(1.0)} - 0.05 = 0.320$$

$$S_{LT} = 1629.5 \times 0.320 = 521.4 pcu/h$$

$$K_L = \frac{S_T}{S_L} = \frac{1629.5}{521.4} = 3.125$$

Calculate the total equivalent flow (Table 3.17):

$$q'_T = K_L q_L + q_T = 3.125(40) + 266 + 18 = 490.0 pcu$$

$$F_{TL} = \frac{(q_L + q_T)}{q'_T} = \frac{(40 + 266 + 18)}{490.0} = 0.792$$

$$S_{LTR} = 1629.5 \times 0.792 = 1290.6 pcu/h$$

$$\text{Capacity} = 1290.6 pcu/h \times \frac{21}{60} = 451.7 pcu/h$$

$$V/C = \frac{40 + 266 + 18}{451.7} = 0.717$$

Southbound

$q_R = 91pcu/h$ $q_T = 574pcu/h$ $q_L = 84pcu/h$

(No geometric adjustments - Follow Steps in Figure 3.2 on page 3-19)

Determine S_L

$$q_{opp} = NBT + NBR = (129 - 60) + 668 + 37 = 774pcu \text{ in two lanes}$$

$$q'_{opp} = q_{opp} \times c/g_e = 774 \times 60/33 = 1407.3pcu$$

$$q'_{ped} = 42 \times 60/32 = 79ped/h \text{ Ignore since <200}$$

$$F = 0.625 \text{ (Table 3.16)}$$

$$F_L = 105e^{(-0.00121)(1407.3)(0.625)} - 0.05 = 0.312$$

$$S_{LT} = 1700 \times 0.312 = 530.4pcu/h$$

Determine S_R

$$q'_{ped} = q_{ped} \times c/g_e = 125 \times 60/32 = 234pcu/h$$

$$F_{RPed(Edmon)} = 0.44 - 234/9320 = 0.415$$

$$S_R = 1700 \times 0.415 = 705.5pcu/h$$

Calculate turning factors

$$K_L = \frac{S_T}{S_L} = \frac{1700}{530.4} = 3.205$$

$$K_R = \frac{S_T}{S_R} = \frac{1700}{705.5} = 2.41$$

Calculate equivalent through flows (Figure 3.2 Step b)

$$q'_L = q_L K_L = (84 - 60) \times 3.205 = 76.9$$

$$q'_R = q_R K_R = 91 \times 2.41 = 219.3$$

$$q'_T = q_T \times 1.0 = 574$$

$$q'_L + q'_R + q'_T = 870.2$$

In 2 lanes, therefore $q'_{Lane} = 870.2 / 2 = 435.1$

Allocation of Equivalent through flows (Figure 3.2 Step d)

$$q'_{TL} = q'_{Lane} - q'_L = 435.1 - 76.9 = 358.2pcu$$

$$q'_{TR} = q'_{Lane} - q'_R = 435.1 - 219.3 = 215.8pcu$$

Conversion back to flows (Figure 3.2 Step e)

$$q_{TL} = q'_L / K_L + q'_{TL} = 76.87/3.205 + 358.2 = 382.2 \text{ pcu/h}$$

$$q_{TR} = q'_R / K_R + q'_{TR} = 219.3/2.41 + 215.8 = 306.8 \text{ pcu/h}$$

Shared through/right lane (Table 3.19)

$$q'_T = q'_{Lane} = 435.1 \text{ pcu}$$

$$F_{TR} = (q_R + q_T)/q'_T = (91 + 215.8)/435.1 = 0.705$$

$$S_{TR} = 1700 \times 0.705 = 1199 \text{ pcu/h}$$

$$C_{TR} = 1199 \times (33/60) = 659 \text{ pcu/h}$$

$$V/C \text{ or } x = (91 + 215.8)/659 = 0.465$$

Southbound through/left Lane (Figure 3.17)

$$q'_T = q'_{Lane} = 435.1 \text{pcu}$$

$$F_{TL} = (q_L + q_T)/q'_T = (84 - 60 + 358.2)/435.1 = 0.878$$

$$S_{LT} = 1700 \times 0.878 = 1492 \text{ pcu/h}$$

$$L_{TOI} = (3600/60) = 60 \times 1 = 60$$

$$C_{TL} = 1492.6 \times 33/60 + (3600/60 \times 1) = 880.9 \text{ pcu/h}$$

$$\text{Clearing volume for left} = 84 - 60 = 24$$

$$V/C = (358.2 + 24 + 60)/880.9 = 0.502$$

Northbound

$q_R = 37$pcu/h $q_T = 668$ pcu/h $q_L = 129$ pcu/h

(No geometric adjustments - Follow Steps in Figure 3.2 on page 3-19)

Determine S_L

$$q_{opp} = SBT + SBR = 574 + 91 + (84 - 60) = 689 \text{ pcu/h}$$

$$q'_{opp} = q_{opp} \times c/g_e = 689 \times 60/33 = 1252.7 \text{pcu/h}$$

Only 2 pedestrians per cycle, so pedestrians' influence on left turns can be ignored (See page 3-44).

$$F = 0.625 \text{ (Table 3.16)}$$

$$F_L = 105e^{(-0.00121)(1209.09)(0.625)} - 0.05 = 0.357$$

$$S_L = 1700 \times 0.357 = 606.9 \text{ pcu/h}$$

Determine S_R

$$q'_{ped} = q_{ped} \times c/g_e = 42 \times 60/33 = 76 < 200 \text{ Therefore, no adjustment}$$

Since there is no adjustment $S_R = S_T = 1700$ pcu/h

Calculate turn factors

$$K_L = \frac{S_T}{S_L} = \frac{1700}{606.9} = 2.80$$

$$K_R = \frac{S_T}{S_R} = \frac{1700}{1700} = 1.00$$

Calculate equivalent through flows (Figure 3.2 Step b)

$$q'_L = q_L K_L = (129 - 60) \times 2.80 = 193.2 \text{ pcu/h}$$

$$q'_R = q_R K_R = 37 \times 1.0 = 37 \text{ pcu/h}$$

$$q'_T = q_T \times 1.0 = 668 \times 1.0 = 668 \text{ pcu/h}$$

$$q'_L + q'_R + q'_T = 898.2 \text{ pcu/h}$$

In 2 lanes, therefore $q'_{Lane} = 898.2/2 = 449.1$ pcu/h

Allocation of Equivalent through flows (Figure 3.2 Step d)

$$q'_{TL} = q'_{Lane} - q'_L = 449.1 - 193.2 = 255.9 \text{ pcu/h}$$

$$q'_{TR} = q'_{Lane} - q'_R = 449.1 - 37 = 412.1 \text{ pcu/h}$$

Conversion back to flows (Figure 3.2 Step e)

$$q_{TL} = q'_L/K_L + q'_{TL} = 193.2/2.80 + 255.9 = 324.9 \text{ pcu/h}$$

$$q_{TR} = q'_R/K_R + q'_{TR} = 37/1 + 412.1 = 449.1 \text{ pcu/h}$$

Shared through/right lane (Table 3.19)

$$q'_T = q'_{Lane} = 449.1$$

$$F_{TR} = (q_R + q_T)/q'_T = (37 + 412.1)/449.1 = 1.0$$

$$S_{TR} = 1700 \text{ pcu/h}$$

$$C_{TR} = 1700 \times (33/60) = 935 \text{ pcu/h}$$

$$V/C \text{ or } x = 449.1/935 = 0.48$$

Shared through/left Lane (Figure 3.17)

$$q'_T = q'_{Lane} = 449.1$$

$$F_{TL} = (q_L + q_T)/q'_T = (129 - 60 + 255.9)/449.1 = 0.723$$

$$S_{LT} = 1700 \times 0.723 = 1229.1 \text{ pcu/h}$$

$$L_{TOI} = (3600/60) \times 1 = 60$$

$$C_{TL} = 1229.1 \times 33/60 + 3600/60 \times 1 = 736.0 \text{ pcu/h}$$

$$\text{Clearance left volume} = 129 - 60 = 69 \text{ pcu/h}$$

$$V/C = (255.9 + 69 + 60)/736 = 0.523$$

6.4.3 Summary

Table 6.4.1 Summary of the Example

Movement		# of lanes	Phase	Q_{adj} pcu/h	RTOI/ LTOI	q cleared pcu/h	S_{adj} pcu/h	y	Critical y	g_e sec	Total C	x	d_1 sec	d_2 sec	d sec	Avg Q_{cons} pcu/ lane
SB	**TR**	1	1	307	0	307	1200	0.256		33	660	0.465	8.16	2.35	11	5
	TL	1	1	442	60	382	1493	0.256		33	881	0.502	8.39	2.04	10	7
NB	**TR**	1	1	449	0	449	1700	0.264	0.264	33	935	0.480	8.26	1.77	10	7
	TL	1	1	385	60	325	1230	0.264		33	736	0.523	8.53	2.65	11	6
WB	**TRL**	1	2	302	0	302	1221	0.247		21	427	0.707	16.84	9.50	28	5
EB	**TRL**	1	2	324	0	324	1290	0.251	0.251	21	452	0.717	16.92	9.39	28	5
								Overall	0.515							

Overall Level of Service (See: Section 4.7.4 Level of Service on page 4-99)

$$\frac{V}{C_{overall}} = \Sigma\frac{q_\phi}{S_\phi} \times \frac{c}{g_e}$$

where $\Sigma\frac{q_\phi}{S_\phi}$ = the sum of the critical q/S values for each phase

$$\frac{V}{C_{overall}} = 0.515 \times (60/54) = 0.57$$

LOS = A

APPENDIX

A

TERMS, SYMBOLS, DEFINITIONS AND ESSENTIAL EQUATIONS

Section A.1 , "Symbols and Equations" on page A-2 lists the basic symbols used for various terms with their units and important equations. Additional notations in the text employ self-explanatory indices.

Section A.2 , "Definitions" on page A-5 provides definitions. The terms are organized in alphabetical order, followed by the symbol, the definition and the number of the principal relevant Section. The symbols and definitions conform as closely as possible to the pioneering work by Webster (Webster and Cobbe 1966) and the chapter on traffic signals in the Manual of Uniform Traffic Devices for Canada (TAC 1998). The Ontario Traffic Manual Book 12 (Ontario 2001) and the Highway Capacity Manual (TRB 2000) have also been used as a source for some symbols and definitions.

"Interval" is used to describe the time during which the signal indication is displayed. "Effective green interval" is a practical exception. "Period" may consist of a combination of intervals, such as "intergreen period" or portions of intervals, such as "all-red period". It is also used for the "analysis period" and the "period of congestion". The term "time" is reserved for "cycle time", "lost time", "evaluation time" and "transit assessment time". "Length" is applied only to distances; for example, "queue length".

A.1 Symbols and Equations

Table A.1 Symbols and equations

SYMBOL	TERM	UNITS	EQUATION
a	deceleration rate	m/s^2	
A	amber period	s	t_{pr} +v / (2a+2gG)
c	cycle time	s	Σ_j (g + A+ $r_{all})_j$ sum over phases j
C	lane capacity	veh/h pcu/h	S g_e / c
$\mathbf{d_o}$	average overall delay	s/pcu s/veh	$K_f\, d_1 + d_2$
$\mathbf{d_1}$	uniform component of average delay	s/pcu s/veh	$c\,(1 - g_e/c)^2 / L_2\,\{1 - x\,(g_e/c)\}$
$\mathbf{d_2}$	overflow component of average delay	s/pcu s/veh	$15t_e\{(x\text{-}1)$ + $+ [(x\text{-}1)^2 + 240 / C\, t_e]^{0.5}$
$\mathbf{d_{o\ int}}$	average overall intersection delay	s/pcu s/veh	$\Sigma_j\, \Sigma_i\, q_{ij} d_{oij} / \Sigma_j\, \Sigma_i\, q_i$ sum over lanes i and phases j
$\mathbf{d_{ped}}$	average delay to pedestrians	s/ped	$(c\text{-}w)^2$ / 2c
$\mathbf{D_o}$	total overall delay	s, h	q d_o
exp	base of natural logarithms		2.71828
e	unit of pollutant emissions	g/s g/stop g/ 100 m	
E	total emissions of a given pollutant	kg	
EHV	passenger car equivalent for heavy vehicles from the Highway Capacity Manual	pcu	
f	factor or coefficient		
F	saturation flow adjustment factor		
g	gravity constant	m/s^2	9.81
g	displayed green interval	s	
$\mathbf{g_e}$	effective green interval	s	g + 1
G	grade (slope) of an approach	% %/100	
HV	percentage of heavy vehicles		
i	summation counter		
int	subscript for “intersection”		
I	intergreen period	s	

Table A.1 Symbols and equations

SYMBOL	TERM	UNITS	EQUATION
j	summation counter		
k	various coefficients or counters		
K	factor used to determine the equivalent through flow		S_T / S_L or S_T / S_R
l	lost time for a single phase	s	(I - 1)
L	total lost time in a single cycle	s	$\Sigma_j (I - 1)_j$ sum over all phases j
L	length or distance	m	
L	subscript indicating left turning		
n	number of cycles		
N	number of stops		
opp	subscript denoting opposing flow to left-turn flow		
O	vehicle occupancy	person	
OF	Overload Factor		
P	probability, with its argument in ()		
q	average vehicular arrival flow in a lane	pcu/h pcu/s veh/h veh/s	
$\mathbf{q_{ped}}$	pedestrian flow in a crosswalk in both directions	ped/h	
$\mathbf{q_{person}}$	average arrival person flow	person/h	$\Sigma_k q_k O_k$ sum over vehicle categories k
Q	various types of queues further denoted by subscripts	pcu m	
r	displayed red interval	s	c - g
$\mathbf{r_{all}}$	all-red period	s	
$\mathbf{r_e}$	effective red interval	s	$c - g_e$
R	subscript indicating right turning		
S	saturation flow in a lane	pcu/h veh/h	
t	time periods further specified by a subscript		
$\mathbf{t_a}$	transit assessment time	min	
$\mathbf{t_{cped}}$	pedestrian clearance period	s	W_{ped} / v_{ped}
$\mathbf{t_e}$	evaluation time	min	
$\mathbf{t_{pr}}$	perception and reaction time	s	

Table A.1 Symbols and equations

SYMBOL	TERM	UNITS	EQUATION
T	subscript for straight-through movement		
u	unit fuel consumption	g/s g/stop g/ 100m	
$\mathbf{u_L}$	ratio of available to required queueing or discharge distance		$u_L = L_a / L_r$
U	total fuel consumption	kg L	
v	speed	m/sec km/h	
V	volume	pcu/h veh/h	
w	walk interval	s	
W	width of lane, intersection, roadway, etc.	m	
x	degree of saturation		q / C
X	number of arrivals in a lane during a given cycle	pcu veh	q c / 3600
$\mathbf{X_{cap}}$	lane capacity during one cycle	pcu veh	C c / 3600
y	lane flow ratio		q / S
Y	intersection flow ratio		$\sum_j y_{j\ critical}$ sum over all phases j

A.2 Definitions

adjustment factor (F): a multiplicative factor applied to the basic saturation flow to adjust it to prevailing conditions.

all-red period (r_{all}): the period during which all traffic signal heads display red signal indications. Signal indications for movements continuing in the following stage may display green.

***amber interval* (A):** that portion of the signal cycle during which traffic facing a circular or arrow amber signal indication must stop before the stop line or other legally defined intersection boundary, unless such stop cannot be made safely. Note that some jurisdictions may use different definitions.

approach (also intersection approach): a section of the roadway upstream of the intersection stop line in which queues form. A minimum length is taken as 50 m.

arrival flow (q): flow rate on an intersection approach lane (exceptionally on a combination of lanes) upstream of the queue influence.

arrival overflow: an operational condition in which the number of vehicles arriving in a single cycle exceeds cycle capacity.

bicycle: non-motorized two- or three-wheeled vehicle (it includes tricycles)

capacity (C): maximum departure flow that can discharge across the stop line of an intersection lane over an extended period of time, usually not less than 15 minutes (exceptionally across a stop line of an approach or a combination of approach lanes). *Cycle capacity* refers to such capacity divided by the number of cycles during the period considered.

conflicting flow: the flow of traffic that is in a potential conflict with a specific movement.

congestion period: the period of time (longer than just a few cycles) during which a continuous queue exists.

control conditions: prevailing conditions concerning traffic controls and regulations in effect, including the type, phase composition, cycle structure and timing of traffic control signals, stop and yield signs, permitted and prohibited movements and similar measures.

coordination of signals: linking of traffic control signal timing at adjacent intersections in order to achieve specific operational objectives, such as progressive movement of traffic or queue control.

critical lane: the lane with the highest flow ratio for a given phase.

crosswalk: a designated portion of the roadway for the use of pedestrians.

cycle (also signal cycle): one complete sequence of signal indications for all phases.

cycle time (c): duration of one cycle.

cycle structure: the sequence and composition of phases in one cycle. A set of detailed cycle structure illustrations is included in the Manual of Uniform Traffic Control Devices for Canada (TAC 1998).

degree of saturation (x): ratio of arrival flow to capacity.

delay: the difference between the time required by an unimpeded passage of a flow unit through the intersection and the time actually needed under the prevailing geometric, traffic and control conditions. The following types of delay are used in the Guide:

- **overall delay:** the time difference includes the time required for acceleration and deceleration.
- **stopped delay:** the time difference includes only the time when the vehicles were stopped or were moving at speeds lower than walking speed in a queue. Note that this defini-

tion differs from the Highway Capacity Manual definition.

- **total overall delay (D_o):** the sum of overall delays experienced by the flow units (passenger car units, vehicles or persons).
- **total stopped delay (D_s):** the sum of stopped delays experienced by the flow units (passenger car units or vehicles).
- **average overall delay (d_o):** total overall delay divided by the number of flow units (passenger car units or vehicles).
- **average stopped delay (d_s):** total stopped delay divided by the number of flow units (passenger car units or vehicles).
- **average pedestrian delay (d_{ped}):** average delay encountered by pedestrians at a crosswalk.
- **average uniform delay (d_1):** that portion of vehicular delay which can be calculated from the uniform arrival flow at undersaturated conditions.
- **average overflow delay (d_2)** which consists of:
  - **random overflow delay:** that portion of vehicular delay which occurs during random cycle overflows, and
  - **continuous overflow delay:** that portion of vehicular delay which occurs while arrival flow exceeds capacity during many consecutive cycles.

departure flow: the rate at which the vehicles (or passenger car units) cross the stop line of a given lane during a given portion of the cycle time. Its maximum value is the saturation flow.

discharge overflow: a situation that occurs when vehicles are unable to depart within the cycle in which they arrived.

display: see signal indication.

don't-walk interval: that part of the cycle during which a steady or flashing red hand signal indication is displayed for a given crosswalk.

downstream: the direction in which traffic is flowing.

driver: the person responsible for the controlling of a vehicle, including bicycle and motorcycle riders.

dwell time: the time that a transit vehicle spends at a stop or station in order to discharge and board passengers.

effective green interval (g_e): duration of time equivalent to the period during which the departure flow of a fully saturated green interval can be represented by a uniform saturation flow.

emission rate (e): individual pollutants emitted from an average passenger car per second of idling or during acceleration to a given speed following a full stop.

evaluation time (t_e): the period of time with a constant (or approximately constant) average arrival flow.

exclusive lane: an approach lane dedicated to only one direction of a departure movement (typically left-turn movement, straight-through movement or right-turn movement)

fixed-time signal operation: a control mode of a signalized intersection during which the sequence and duration of all signal indications (timing program) remains unchanged.

fixed-cycle signal operation: a control mode of a signalized intersection during which the cycle time remains constant but the sequence and duration of some or all signal indications may vary in response to traffic demand.

flow: see arrival or departure flow.

flow ratio (y): ratio of arrival flow and saturation flow.

gap: the time between two successive vehicles in a lane as they pass a point on the roadway, measured from the rear of the first vehicle to the front of the following vehicle.

geometric conditions: spatial characteristics of a road or an intersection, including the type of facility, number and width of lanes, shoulders and sidewalks, radii, grades and other features of the horizontal and vertical alignments. The term normally includes lane allocation to directional flows designated by pavement markings and signs.

green interval: that portion of the signal cycle during which traffic facing the circular or arrow green signal indication may proceed through the intersection in accordance with local laws and rules of the road.

headway: the time between two successive vehicles in a lane as they pass a point on the roadway, measured from the front of one vehicle to the front of the successive vehicle.

interval: the duration of time during which a given signal indication is displayed.

intergreen period (I): the duration of time separating the end of the displayed green interval from the beginning of the next conflicting displayed green interval.

level of service: a qualitative measure used to describe operational conditions within a traffic stream, dependent on vehicle throughput and based on volume capacity ratio and also generally reflecting such factors as speed, travel time, delay, and freedom to manoeuvre.

left-turn movement: a legally permitted movement of a vehicle which must cross the potential path of vehicles in the opposing direction. For signal indication examples refer to the Manual of Uniform Traffic Control for Canada (TAC 1998).

lost time in a phase (l): a portion of the phase (defined as the actual green interval plus the following intergreen period) that remains after the effective green interval has been subtracted.

lost time in a cycle (L): the sum of lost times for all consecutive phases in the cycle.

movement or intersection movement: any legally permitted movement of a vehicle from a given lane.

overflow queue: a residual queue at the end of a cycle waiting to be discharged in the subsequent cycle.

overload factor (OF): the number of overloaded cycles divided by the total number of surveyed cycles.

overloaded cycle: a cycle in which the number of vehicles that arrived during that cycle plus the number of vehicles present at the beginning of that cycle in the lane under consideration could not fully discharge. Some vehicles had to wait for the subsequent cycle in an *overflow queue*.

passenger car: a motorized four-wheeled vehicle designed primarily for the transport of up to nine passengers. The term normally includes pickup trucks and vans with no more than four tires.

passenger car unit (pcu): a unit of completely homogeneous traffic, represented in practical terms by an average passenger car.

pedestrian: a person afoot, in a wheelchair or pushing a bicycle.

pedestrian clearance period (t_{cped}): the minimum duration of time needed for a pedestrian who entered the crosswalk at the very end of the walk interval to reach a refuge in reasonable comfort.

permissive left turn: a left-turn movement that take place while the drivers in the opposing direction of traffic face a circular green indication. Drivers making this left turn must yield the right-of-way to the opposing flow.

person: a driver or a passenger in a vehicle, but not a pedestrian.

person flow: the number of persons moving through an intersection on or inside vehicles. It does not include pedestrians.

phase: that a portion of a cycle during which the allocation of the right of way remains unchanged. This term normally includes the associated intergreen period. For full descriptions of terms, such as *full phase, leading left-turn phase, leading phase, lagging phase, simultaneous left-turn phase* and some of their combinations, including the sequences of signal indications, refer to the Manual of Uniform Traffic Control Devices (TAC 1998).

phase composition: the combination of vehicular, pedestrian and other movements legally permitted during a phase.

phase sequence: the order in which the phases follow each other in a cycle.

platoon: a line of relatively fast moving vehicles with high traffic density (see also *queue*).

progression: a continuous movement of traffic in a given direction through two or more signalized intersections.

protected left turn: a left-turn movement that take place while the drivers in the opposing direction of traffic face a circular red indication.

queue: a line of traffic units (vehicles or pedestrians) waiting to be served by a signalized intersection. Slowly moving traffic joining the rear of the queue are usually considered a part of the queue. The internal queue dynamics may involve a series of stops and starts. A faster moving line of traffic is often referred to as a moving queue or a *platoon.*

queue length (Q): the number of traffic units in a queue, or the distance which is covered by the queue.

queue reach (Q): used for vehicular traffic only, defined as the distance between the stop line of a lane and the point upstream at which vehicles are joining the queue, expressed as the number of vehicles that would fill that distance or in metres. The front of the queue may be some distance upstream from the stop line.

saturation flow (S): the departure rate from a queue during the green interval measured at the stop line.

- **basic saturation flow:** the departure rate from a queue in a 3.0 to 4.0 m wide lane that carries only straight through passenger car traffic, and is unaffected by conditions such as grade, parking, etc. Vehicles are considered discharged when their fronts cross the stop line. Note that the Highway Capacity Manual employs a different definition and measurement method from the Guide.
- **adjusted saturation flow:** saturation flow adjusted for the effect of specific local conditions or measured under conditions different than those specified for basic saturation flow.

shared lane: a lane from which vehicles may discharge in more than one downstream direction.

signal indication: the following signal indications are defined in detail in the Manual of Uniform Traffic Control Devices for Canada, (TAC 1998): green, amber, red, circular, arrow, steady, flashing, walk, don't-walk, flashing don't-walk and transit priority signal indications.

signal operation: a term to describe the function of a signalized intersection, such as in the following:

- **fixed-time operation:** a pre-programmed sequence and duration of signal indications. All signal intervals remain constant and are not affected by variations in traffic flow.

- **fixed-cycle operation:** a set of pre-programmed rules that determine the sequence and duration of signal phases or intervals within a cycle of fixed duration.

traffic responsive operation: any of the control modes that adjust the signal timing to prevailing traffic conditions: The following types are included:

- **fully traffic actuated:** a set of pre-programmed rules determines the sequence of all signal intervals with their duration extending from fixed minima to fixed maxima based on gap-seeking traffic measurements.
- **semi-traffic actuated:** a set of pre-programmed rules allows the initiation of some signal intervals based on the presence of vehicles or a pedestrian actuation, possibly with extensions from fixed mimima to fixed maxima based on gap-seeking traffic measurements.

simultaneous left turn: two opposing left-turn movements that take place simultaneously while drivers of both opposing straight-through traffic directions face circular red indications.

traffic adaptive operation: a set of algorithmic rules that adjust all facets of intersection control in accordance with one or more objective functions and constraints.

traffic: the movement of motorized and non-motorized vehicles and pedestrians.

traffic conditions: the combination of pedestrians and vehicle types at the intersection and on adjacent roadways, sidewalks, bicycle and other traffic facilities, including the temporal, directional and lane use distributions of traffic, and the types of driver or other user population.

traffic flow: see *arrival* or *departure flow*.

transit assessment time: the period of constant or nearly constant occupancy of transit vehicles.

transit priority: a control mode in which transit vehicles receive a signal indication that provides for some advantage (usually shorter delay) to transit operations.

truck: a heavy vehicle designed primarily for the transport of goods.

upstream: the direction from which traffic is coming.

volume (V): the number of persons or vehicles passing a point on a roadway lane in a given time period.

walk interval (w): the duration of display of the walk signal indication.

Appendix B

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