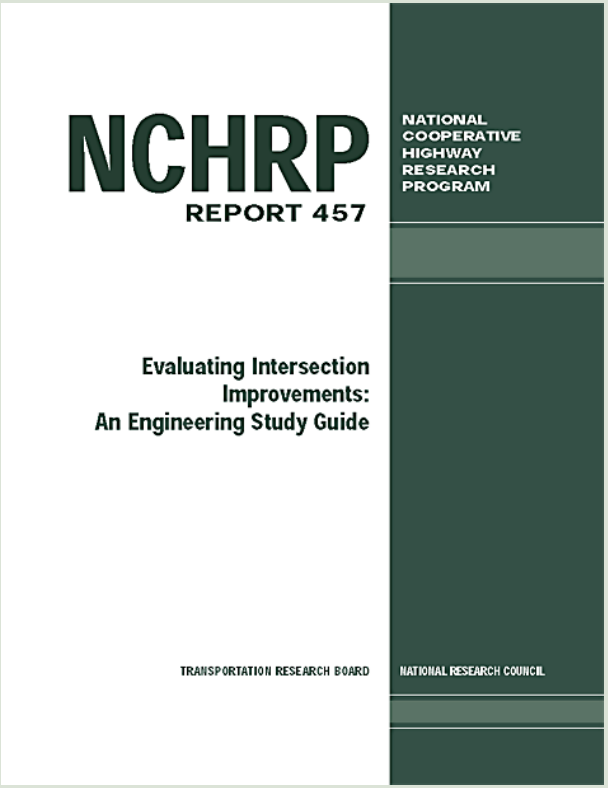

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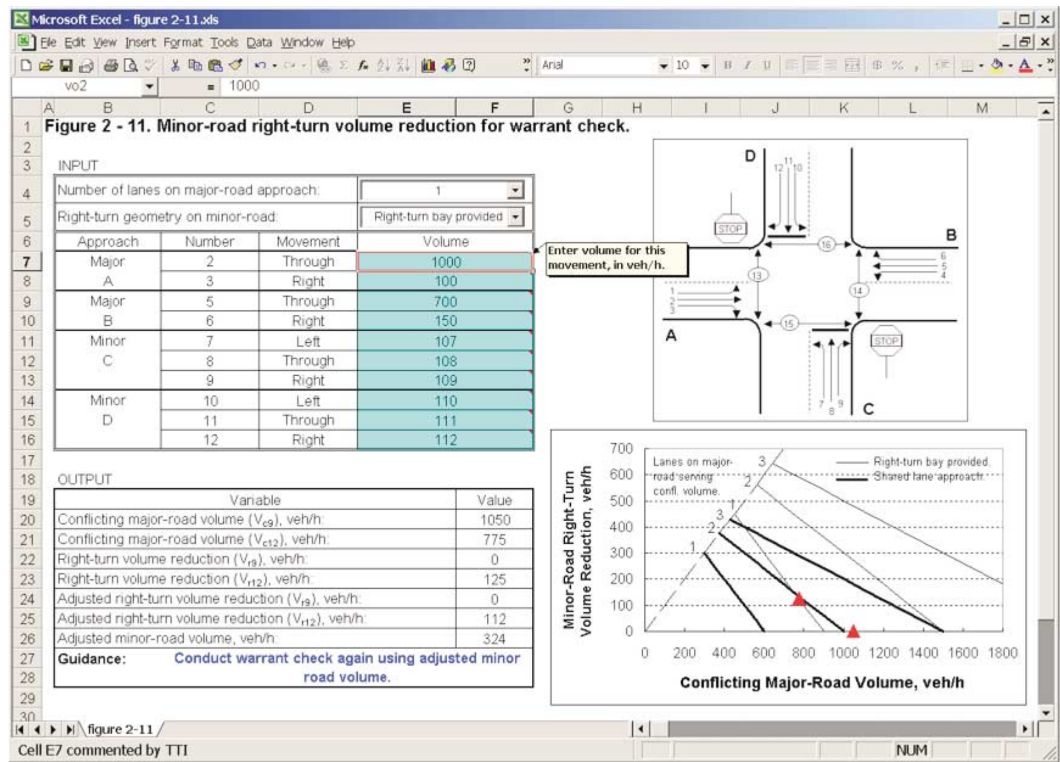

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NCHRP

REPORT 457

NATIONAL
COOPERATIVE
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RESEARCH
PROGRAM

Evaluating Intersection Improvements: An Engineering Study Guide

TRANSPORTATION RESEARCH BOARD

NATIONAL RESEARCH COUNCIL

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 457

Engineering Study Guide for Evaluating Intersection Improvements

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College Station, TX

SUBJECT AREAS

Highway Operations, Capacity, and Traffic Control

Research Sponsored by the American Association of State Highway and Transportation Officials
in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD — NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS

WASHINGTON, D.C. — 2001

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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NCHRP REPORT 457

Project G3-58 FY'99

ISSN 0077-5614

ISBN 0-309-06705-7

Library of Congress Control Number 2001-131981

Price $40.00

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The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

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Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

and can be ordered through the Internet at:

http://www.national-academies.org/trb/bookstore

Printed in the United States of America

FOREWORD

By Staff
Transportation Research Board

This guide describes the engineering study process for evaluating the operational effectiveness of various intersection improvements. It also shows how capacity analysis and traffic simulation models can be used to assess the operational impacts of those improvements. Use of this guide, particularly by junior traffic engineers, should enhance the decision-making process and reduce inappropriate installations of traffic control signals. An enhanced version of this report is available on the world wide web at http://trb.org/trb/publications/nchrp/esg.pdf.

The *Manual on Uniform Traffic Control Devices-Millennium Edition (MUTCD 2000*) states as a standard that an “engineering study of traffic conditions, pedestrian characteristics, and physical characteristics of the location shall be performed to determine whether installation of a traffic control signal is justified at a particular location.” The *MUTCD 2000* furthers states as guidance that “a traffic control signal should not be installed unless an engineering study indicates that installing a traffic control signal will improve the overall safety and/or operation of the intersection.” The *MUTCD 2000* describes some aspects of the engineering study, including the traffic signal warrants, but does not attempt to fully describe the decision-making process.

Capacity analysis (e.g., the methods in the TRB *Highway Capacity Manual*) and traffic simulation models can be beneficially used in these engineering studies to assess the operational impact of a traffic control signal and other intersection improvements. Sometimes these tools may show that, while the warrants are met at a particular location, a less costly improvement would operate more effectively than a traffic control signal.

Under NCHRP Project 3-58, the Texas Transportation Institute analyzed difficulties commonly faced when using traffic signal warrants to determine the appropriateness of a traffic control signal. They also identified operational measures of effectiveness that should be considered in the assessment of intersection improvements. They then developed a guide to conducting an engineering study of an intersection.

Evaluating Intersection Improvements: An Engineering Study Guide defines the steps involved in an engineering study of a problem intersection, beginning with identifying the problem and viable alternative improvements to address the problem. It also illustrates how to use capacity analysis and traffic simulation models to determine the most effective operational improvement. The guide does not assist in the analysis of the safety or other impacts of the alternative improvements, although these must be considered when determining the most appropriate improvement. References to other sources of information on these types of analysis are provided.

The reader may be interested in an enhanced version of the guide available on the web. That version includes internal hyperlinks between different parts of the report and external links to source material. This web version also includes 17 interactive worksheets that can be helpful in using the guide.

CONTENTS

AUTHOR ACKNOWLEDGMENTS

The research team would like to recognize the support and guidance provided by the project panel. The project panel members include Dr. Robert Wortman (chair), University of Arizona; Mr. Rick Bennett, Missouri Department of Transportation; Dr. Thomas H. Culpepper, Wilbur Smith and Associates; Ms. Beth P. De Angelo, Parsons Brinkerhoff—FG, Inc.; Mr. Glenn M. Grigg, consultant; Mr. Para M. Jayasinghe, City of Boston, Massachusetts; Mr. Steve G. Jewell, City of Columbus, Ohio; Mr. Ken Kobetsky, AASHTO; Mr. Pawan Maini, Fitzgerald and Halliday, Inc.; and Mr. Henry Lieu, Federal Highway Administration.

Finally, the research team would like to acknowledge the engineers who participated in the focus group assembled for the research project. These individuals include Mr. W. Martin Bretherton, Jr., Gwinnett County Department of Transportation, Georgia; Mr. Ron Northouse, City of San Jose, California; Mr. Michael R. Scanlon, City of Tampa, Florida; Mr. Richard Filkins, Washington State Department of Transportation; and Mr. Chad J. Smith, Iowa Department of Transportation. Their thoughtful advice and comment on the procedures and guidelines are greatly appreciated.

The **Transportation Research Board** is a unit of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. The Board's mission is to promote innovation and progress in transportation by stimulating and conducting research, facilitating the dissemination of information, and encouraging the implementation of research results. The Board's varied activities annually draw on approximately 4,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation.

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation

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CHAPTER **1**

INTRODUCTION

The *Manual on Uniform Traffic Control Devices-Millennium Edition* (*MUTCD 2000*) (*1*) includes warrants that set minimum thresholds for considering the installation of a traffic signal. The threshold values used in these warrants were established many years ago. They are based largely on engineering judgment and reflect sensitivity to two variables: approach volume and number of lanes. However, their subjective basis and limited sensitivity can make them inaccurate predictors of the need for signal control at some intersections and could result in the unnecessary installation of traffic signals. In fact, two separate evaluations of the *MUTCD* warrants found that they do not always yield conclusions that agree with engineering judgment (*2*,*3*). This finding is a concern because a recent literature review by Bonneson and Fontaine (*4*) revealed that delays and stops can increase 100 to 200 percent when marginally warranted signals are installed at an intersection.

To eliminate the unnecessary installation of traffic signals, the text of the *MUTCD* was revised for its 1988 publication. The purpose of the revisions was to indicate clearly that the decision to install a traffic signal should be based on the findings from an engineering study. The 1998 *MUTCD* stated that "satisfaction of a warrant or warrants is not in itself justification for a signal." This text was retained for the *MUTCD 2000* and further states that "A traffic control signal should not be installed unless an engineering study indicates that installing a traffic control signal will improve the overall safety and/or operation of the intersection." The intent of this recommendation is to encourage consideration of the full range of intersection improvement alternatives and selection of the most effective alternative for implementation.

This guide documents the steps involved in the formal engineering study of improvement alternatives and focuses on the use of capacity analysis procedures and simulation models (i.e., analysis tools) to evaluate the operational impacts of improvement alternatives. Case studies illustrate how these analysis tools can sometimes be used to show that an intersection would operate more effectively without a traffic signal, even though a signal warrant is met.

OVERVIEW OF THE ASSESSMENT PROCESS

Generally, the engineering assessment process is initiated when operational or safety problems are identified at an unsignalized intersection. The goal of this process is to identify the most effective solution to the identified problem. The assessment process is often integral to a larger, more comprehensive system management process in which all problematic transportation facilities are considered for improvement. The stages of the system management process are shown in Figure 1-1.

The assessment process represents three specific stages of the system management process. In the first stage, viable alternatives are identified. In the second stage, an engineering study is conducted to evaluate the effectiveness of each viable alternative. Finally, the best alternative is selected on the basis of its effectiveness and its other, non-motorist-related effects.

Following the assessment process, the best alternative for a given intersection is pooled with other improvement projects for funding consideration. If funding is available, the alternative is constructed, and the intersection is monitored to confirm that the original problem has been solved.

Alternative Identification and Screening

During the alternative identification and screening stage, the traffic engineer makes a preliminary assessment of the problem and identifies viable intersection improvement alternatives. Alternatives may include changes in traffic control, intersection geometry, or both. Traffic control alternatives can include two-way stop control, multi-way stop control, or signal control. Geometric alternatives that are sometimes identified include the addition of left or right-turn bays on the major or minor roadways.

Initially, alternatives are identified that could solve the observed problem. Then, this list of candidate alternatives is reduced, using formal guidelines or warrants, to include only the most viable alternatives. Formally recognized guidelines exist for traffic signals, multi-way stop control, and some major geometric improvements. Guidelines for control device applications are provided in the *MUTCD 2000* (*1*). Similarly, guidelines that describe when a left-turn bay is needed on the major-road approach to an unsignalized intersection are provided in *A Policy on Geometric Design of Highways and Streets* (*Green Book*) (*5*).

Engineering Study

During the engineering study stage, the viable alternatives are evaluated in terms of their effect on road users and the

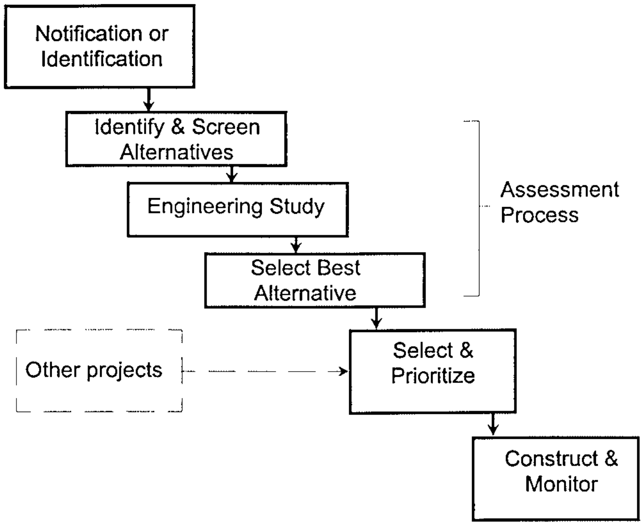


Figure 1-1. Stages of the system management process.

immediate environment. The accurate assessment of these effects typically requires the application of formally recognized analysis procedures. However, in some less-complex situations, experience with similar alternatives and operating conditions may be sufficient to enable the engineer to estimate an alternative's effectiveness.

Alternative Selection

During the alternative selection stage, the engineer assesses the effects of each alternative and then selects the "best" one for implementation. Effects considered may include improvements in traffic operations or safety and disruption to area aesthetics, the environment, or adjacent property. Selection of the best alternative may also reflect consideration of the construction cost of each alternative. The method of selection can vary from a complicated life-cycle, benefit-cost analysis to a simple identification of the alternative that yields the least traffic delay. The methods used and the effects considered will depend on the conditions present at the problem location.

OBJECTIVE AND SCOPE

Objective

This guide has two objectives: (1) to define the steps involved in an engineering study of a problem intersection and (2) to provide guidelines for using capacity analysis or simulation to determine the most effective alternative on the basis of operational considerations. To achieve these objectives, the coverage of the guide is expanded to include the three stages of the assessment process, as shown in Figure 1-1. This approach provides a comprehensive treatment of problem intersections by preceding the engineering study stage with an alternative identification and screening stage and following it with an alternative selection stage. Application of all three stages will ensure that a wide range of alternatives is considered and that the alternative selected is effective.

Scope

The guide is intended to describe how to evaluate the operational effects of alternative geometrics and control modes at a problem intersection. This description includes guidelines on (1) the alternative selection process and (2) the use of capacity analysis procedures and simulation models for alternative evaluation. The assessment of an alternative's safety (or other) effect is beyond the scope of the guide. The analyst is encouraged to consult the References and Bibliography for guidance on safety evaluations.

The target audience of this guide is the junior traffic engineer. Procedures and guidelines provided herein are intended to identify critical decision points and problem-solving techniques for engineers with only a few years of experience. To meet the needs of this intended audience, each topic is covered thoroughly. Sometimes, the material herein may contradict the policies and procedures of the reader's agency. In such instances, agencies may substitute their policies or practices for those in this guide.

This guide is intended to be applicable to intersections with a wide range of control modes. However, the three modes commonly found at U.S. intersections are given greater emphasis in some sections of the document. These modes are as follows:

1. Two-way stop control,
2. Multi-way stop control, and
3. Signal control.

Although the three control modes listed above are emphasized, the procedures in the guide can be used to evaluate other control modes (e.g., no control, two-way yield control, and roundabouts).

OVERVIEW OF THE GUIDE

This guide consists of four chapters and three appendixes. Chapters 2, 3, and 4 describe the three stages of the assessment process, as shown in Figure 1-1. A flowchart diagramming the assessment process is shown in Figure 1-2.

Chapter 2 of the guide describes the alternative identification and screening stage. The chapter describes how to identify the problems at the subject intersection, how to identify candidate improvement alternatives, and how to screen these alternatives so that only the most viable alternatives are evaluated in the engineering study.

Chapter 3 of the guide describes the engineering study stage. This chapter describes the process for evaluating the

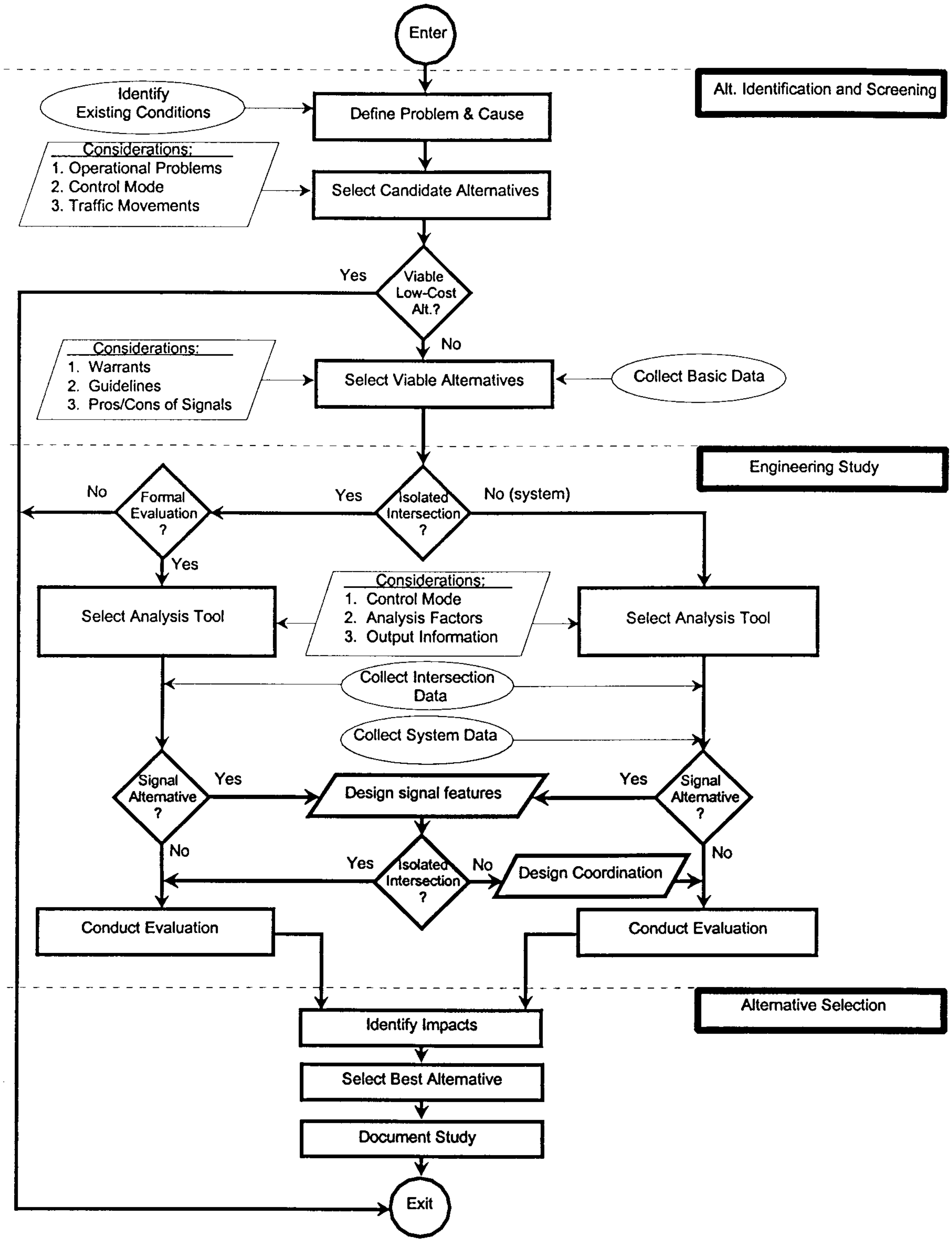


Figure 1-2. Flow chart of the assessment process.

effectiveness of the selected alternatives. The process focuses on how to evaluate traffic operations by using capacity analysis procedures or simulation models. Guidelines are provided to help the analyst determine the most appropriate analysis tool (i.e., capacity analysis procedure or simulation model), the data needed for the evaluation, and whether or not an alternative operates satisfactorily.

Chapter 4 of the guide describes the alternative selection stage, including how to identify the best alternative for a given intersection. Alternative selection is described in general, rather than precise, terms, so as to allow the analyst some flexibility in selecting the types of effects to consider when selecting an alternative and the relative weight to be applied to each effect.

Guidelines that support the assessment process are provided in the Guidelines section of Chapters 2 and 3. The materials provided include (1) a list of traffic control and geometric design alternatives, (2) warrants and guidelines describing conditions suitable for selected alternatives, (3) techniques for designing the traffic signal control alternative, and (4) guidelines for using stochastic simulation models. Use of this guide should help agencies to maximize their return on infrastructure investment and that they will avoid the unnecessary installation of traffic signals.

The appendixes to the guide are intended to provide supplementary information that makes the guide a more versatile document. The use of the guide is demonstrated in several case study situations in Appendix A. Appendix B provides worksheets for documenting the *MUTCD* signal warrant check. Appendix C describes techniques that can be used to gather or derive the data needed for the engineering study.

Although the coverage in the guide is thorough, it is assumed that the reader will have access to two reference documents: (1) the current edition of the *MUTCD* and (2) the *Manual of Transportation Engineering Studies* (*6*). This latter document describes procedures for collecting data for the direct evaluation of intersection performance and for collecting the input data needed for a capacity analysis procedure or simulation model.

Other Impacts

Although the focus of the guide is on evaluating the operational effects of improvement alternatives, the analyst should also consider safety and other effects during the engineering study. Safety impact assessment may range from an informal subjective assessment to a formal quantitative evaluation, depending on the types of problems being experienced (or anticipated) at the subject intersection. Often, the best alternative will be the one that improves traffic operation and enhances safety. However, this may not always be the case—some alternatives may improve operation but degrade safety and vice versa. Therefore, the analyst should carefully evaluate and weigh all relevant effects when selecting the best alternative.

CHAPTER **2**

ALTERNATIVE IDENTIFICATION AND SCREENING

Intersections that are inefficient or unsafe are typically subjected to some type of engineering assessment to identify the underlying problem and its solution. The engineering assessment process consists of three stages: (1) alternative identification and screening, (2) the engineering study, and (3) alternative selection. The first stage of the assessment process is the subject of this chapter. The steps involved in the alternative identification and screening stage are defined in the first section of this chapter. Guidelines are provided in the second section. These guidelines describe (1) conditions where selected improvement alternatives may be helpful and (2) conditions that might mislead the signal warrant check.

PROCESS

Overview

The alternative identification and screening stage consists of three steps. These steps are as follows:

1. Define problem and cause,
2. Select candidate alternatives, and
3. Select viable alternatives.

In the first step, the problem is defined and its cause is identified through the conduct of a site visit and the collection of relevant existing data. Then, several alternatives are selected for further consideration. Finally, these alternatives are screened using available engineering guidelines so that a subset list of viable alternatives is identified.

The objective of the alternative identification and screening stage is to determine if a problem exists and, if it does, to identify one or more viable alternatives that can eliminate or mitigate the problem. This objective is achieved through an evaluation of existing conditions and consideration of a range of improvement alternatives. The steps in this process are described in the remainder of this section.

Step 1. Define Problem and Cause

The first step in the alternative identification and screening stage is to determine the nature of the problem at an intersection and to define its potential causes. The tasks involved in making this determination are to

a. Gather information and
b. Define the problem and cause.

In the first task, data are collected that describe the intersection's history and its present condition in terms of traffic volume, safety record, and geometric layout. Then, these data are used to define the intersection's operational or safety problems and to identify their causes.

The assessment process typically begins when the engineer is notified of a problem at an intersection or series of intersections. This notification can come from sources that are either internal or external to the agency. Some possible sources of problem reports include

- Complaints from local residents about existing intersections,
- Developers seeking traffic control for proposed intersections,
- Observations from field crews,
- Data obtained from consultant studies,
- Information obtained from regional traffic counts,
- Potential problematic conditions identified through regional traffic projections, and
- Annual safety analysis of high-crash locations.

1-a. Gather Information

Overview. Following notification of a problem, the engineer makes a preliminary assessment to determine the extent of the problem and whether it requires a solution. This assessment involves gathering readily available information about the problem intersection so that the problem can be defined and its cause identified. Information sources include historic file data, firsthand observation, and a site survey.

Completion of this task should not require a rigorous data collection effort. A visit to the site should provide sufficient information to determine if a problem exists. Traffic volume, conflict, or delay data should not be needed to make this determination. In Step 3 of the process, Select Viable Alternatives, additional data will be collected to allow a more detailed assessment of the problem and its solution.

Historic Data. The analyst should gather existing engineering studies, corridor studies, traffic studies, and crash data summaries that may help to identify potential problems at the subject intersection. Crash data summarized in a collision diagram may indicate where problems are occurring at the intersection. (Details on the construction of a collision diagram are provided in Appendix C.)

Observational Study. An important component of the problem-cause identification process is the firsthand observation of traffic operations. A visit to observe operations should be scheduled to coincide with the occurrence of the reported problems (e.g., p.m. peak traffic demand hour). Initially, the engineer should drive through the subject intersection and attempt to experience the problem. Then, the engineer should observe intersection operation from a curbside vantage point. An onsite observation report, such as that shown in Figure 2-1, should be completed during the site visit. (Details regarding the completion of this report and a blank report form are provided in Appendix C.)

Site Survey. The engineer should also have a survey of site conditions conducted. The survey data should be recorded on a condition diagram. The diagram represents a plan-view,

ON SITE OBSERVATION REPORT

LOCATION: *Kelly Drive & Tall Trees Lane* DATE: *3/3/00*
CONTROL: *Stop control on Tall Trees Lane* TIME: *4:30 P.M.*

Isolated and Non-Isolated Intersections	No	Not Sure	Yes
1. Do road curvature, vegetation, buildings, parked cars, etc. block drivers' views of conflicting vehicles?	✓		
2. Is the intersection skew angle so sharp that it makes it difficult to view conflicting vehicles or complete turns?	✓		
3. Do vehicle speeds appear too high?		✓	
4. Does the delay for the minor-road right-turn appear excessive?		✓	
5. Does the delay for the minor-road through appear excessive?	✓		
6. Does the delay for the minor-road left-turn appear excessive?			✓
7. Does the delay for the major-road left-turn appear excessive	✓		
8. Does the queue for the major-road left-turn ever impede major-road through traffic?	✓		
9. As major-road vehicles slow to turn, do they impede other vehicles?			✓
10. Do parking maneuvers impede other vehicles?	✓		
11. Are drivers not complying with the traffic control devices?	✓		
12. Is there evidence that one or more curb radii are too small?	✓		
13. Do pedestrians appear to cause conflict with vehicular traffic?	✓		
14. Are there guidance or control problems that could be mitigated by raised-curb channelization?	✓		
Non-Isolated Intersections			
A. Do queues from adjacent signalized intersections spillback into the subject intersection?	*na*		
B. Do vehicles slowing to turn at adjacent intersections or driveways contribute to the delay to major- or minor-road drivers?	*na*		
C. Is it possible that some drivers are diverting to the subject intersection because of congestion on a nearby arterial street?	*na*		
D. Does the arrival pattern of major-road traffic platoons contribute to the delay to minor-road drivers?	*na*		

na = not applicable.

Comments:

Figure 2-1. Sample onsite observation report.

scale drawing of the subject intersection. Conditions recorded may include road width, pavement markings, speed limits, and traffic control devices. (Details of the site survey and a blank condition diagram are provided in Appendix C.)

Traffic Projections. Traffic volume projections are important when a new intersection is being proposed. Projected turning movement counts may provide the best data available for the analyst to determine the most appropriate type of control for the intersection. The analyst should also determine if major changes in traffic patterns or land use are anticipated in the vicinity of the subject intersection. Improvement alternatives should be able to accommodate future traffic patterns.

1-b. Define Problem and Cause

Overview. During this task, the analyst will define the problem and identify its cause. The information gathered in the preceding task is used for this purpose. Initially, the analyst will review this information and determine if sufficient evidence of a problem exists. If it is determined that a problem exists, then the problem is formally defined and its cause is identified.

Assess Evidence. After reviewing the available historical information and information obtained from the site visit, the engineer should determine if sufficient evidence of a problem exists to proceed further in the assessment process. If such evidence exists, then further study may be necessary.

Define Problem and Identify Cause. Problems can generally be categorized as operational or safety-related. Operational problems are typically associated with excessive delay to one or more traffic movements. Safety-related problems are typically associated with frequent conflicts, erratic maneuvers, non-compliance with control devices, and collisions. The information obtained during Task 1-a should be used to determine which (if any) of these two categories of problems exist for each of the intersection traffic movements.

Once the problem has been defined, the engineer should identify what is causing the problem. Problems and potential causes are listed in Table 2-1. The possible causes for each problem category (i.e., delay or conflict) are identified by checkmark (✓). When using Table 2-1, each intersection approach should be individually evaluated. The movements experiencing a problem should be identified and compared with the checked combinations shown. The terms "delay" and "conflict" refer broadly to operational and safety problems observed during the site visit (they are not meant to

TABLE 2-1 Common operational problems and possible causes

Possible Cause	Approach:	Minor-Road						Major-Road					
	Problem:	Excessive Delay			Excessive Conflict[2]			Excessive Delay			Excessive Conflict[2]		
	Movement:[1]	L	T	R	L	T	R	L	T	R	L	T	R
Excessive on-street parking activity.			✓			✓			✓			✓	
Excessive pedestrian volume.		✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Inadequate separation of major-road movements.								✓	✓	✓	✓	✓	✓
Inadequate capacity for minor-road movements.		✓	✓	✓									
Inadequate separation of minor-road movements.		✓	✓	✓									
Inadequate capacity for left-turn movements.		✓						✓					
Inadequate sight-distance along major road.					✓	✓	✓				✓		
Major-road speed too high.					✓	✓	✓				✓		
Inadequate sight-distance along minor road.					✓	✓	✓						
Inadequate protection for pedestrians.					✓	✓	✓				✓	✓	✓
Lack of awareness of Yield sign (driver error).					✓	✓	✓						
Intentional violation of Stop sign.					✓	✓	✓						
Lack of awareness of Stop sign (driver error).					✓	✓	✓						
Excessive volume due to diversion from adjacent roads.		✓		✓	✓		✓	✓		✓	✓		✓
Arrival of major-road platoons is staggered such that there are limited opportunities to enter intersection.		✓	✓	✓				✓					
Queue spillback from downstream signal.		✓	✓					✓	✓	✓			
Queue spillback due to downstream unsignalized left turn.		✓	✓					✓	✓				

Notes:
1 - Movements: L = left-turn, R = right-turn, T = through movement.
2 - Conflicts can include erratic maneuvers, collisions, or non-compliance with control devices.

imply that a delay or conflict study must be conducted before Table 2-1 can be consulted).

To illustrate the use of Table 2-1, consider a minor-road approach at a four-leg intersection. All three movements on this approach were observed to have excessive delay (but no conflicts). Table 2-1 indicates that three possible causes are associated with the observed minor-road delays: (1) inadequate capacity for minor-road movements, (2) inadequate separation of minor-road movements, and (3) staggered arrival of major-road platoons. These possible causes are advanced to the next step to determine candidate improvement alternatives.

Define Influence Area. Designation of the intersection "influence area" is an important part of this task. This area may include the subject intersection only or it may include the network of roads that surround the subject intersection. As a minimum, the influence area should extend from the subject intersection sufficiently far so as to include the queues associated with the intersection's traffic movements.

The extent of the influence area's coverage is based primarily on the proximity of the subject intersection to other intersections. The inclusion of nearby intersections in the influence area would be based in part on the answers to Questions A, B, C, and D on the observation report (Figure 2-1). If one or more of these questions are answered "Yes" or "Not Sure," then the subject intersection may be influenced by one or more nearby intersections.

Efforts to improve conditions at the subject intersection should be sensitive to the operation of all intersections in the influence area. All intersections in the influence area should be included in the evaluation conducted in the engineering study stage (see Chapter 3).

Step 2. Select Candidate Alternatives

After the problem is defined and its likely cause is identified, one or more candidate alternatives should be selected. This section provides a procedure for selecting candidate improvement alternatives. The procedure consists of the following two tasks:

a. Identify potential alternatives and
b. Organize and select alternatives

During the first task, a list of alternatives that could solve the problems is identified. Then, these alternatives are organized and screened to obtain a subset of candidate alternatives based on site constraints and institutional preferences.

2-a. Identify Potential Alternatives

Overview. Tables 2-2 and 2-3 are provided in this section to assist in the identification of potential improvement alternatives. The first table applies to problems that require intersection-specific solutions and the second table applies to intersections requiring system-related solutions. The alternatives listed in these tables emphasize operational improvements; the analyst is referred to Chapter 11 of the *Manual of Transportation Engineering Studies* (*6*) for a complete list of safety problems, causes, and possible solution alternatives.

Intersection-Specific Alternatives. Table 2-2 identifies problems commonly encountered at intersections. This table also lists potential corrective strategies and candidate alternatives. Problems and causes other than those listed may also be known to exist at a specific intersection. In these situations, judgment should be used to identify the appropriate improvement alternatives.

The characteristics of the potential alternatives listed in Table 2-2 are indicated by underline and italic font. Alternatives that affect traffic operations (e.g., motorist delay) are underlined. Alternatives shown in italics should be evaluated for viability in the next step (i.e., Step 3 - Select Viable Alternatives).

To illustrate the use of Table 2-2, consider a minor-road intersection approach observed to have excessive delay caused by inadequate separation of its traffic movements. Table 2-2 indicates that one corrective strategy is to separate the conflicting movements by using one of the following alternatives: (1) add a second lane on the approach or (2) increase right-turn radius. This latter alternative would widen the throat of the approach and effectively separate right-turning vehicles from through and left-turning vehicles.

System-Related Alternatives. Table 2-3 lists the problems that may be found at non-isolated intersections. These intersections have an influence area that extends beyond the limits of the subject intersection and includes the adjacent intersections (either unsignalized or signalized). The improvement alternatives listed in both Table 2-2 and Table 2-3 should be considered when the intersection is not isolated. The alternatives that affect traffic operations are underlined; those alternatives whose viability can be evaluated in the next step are italicized.

2-b. Organize and Select Alternatives

Overview. During this task, the potential alternatives are screened to determine if they are suitable candidates for the subject location and whether their immediate implementation is more cost-effective than the conduct of a formal engineering study. If no alternatives are selected for immediate implementation, the alternatives that remain after screening represent the "candidate" improvement alternatives. These alternatives are then advanced to Step 3.

Identify Candidate Alternatives. Initially, the list of potential improvement alternatives should be screened for

TABLE 2-2 **Potential intersection-specific, engineering improvement alternatives**

Observed Problem	Approach[1] Minor L	Minor T	Minor R	Major L	Major T	Major R	Observed Cause	Corrective Strategy	Potential Alternatives[3]
Delay and Conflict[2]	--	✔	--	--	✔	--	Excessive parking activity.	Reduce activity.	*Prohibit on-street parking during peak hours.* *Prohibit on-street parking permanently.*
	✔	✔	✔	✔	✔	✔	Excessive pedestrian volume.	Separate conflicting flows.	Deploy crossing guard. Relocate crosswalk. *Convert to traffic signal (with pedestrian actuation).*
								Provide guidance.	Add warning signs.
	--	--	--	✔	✔	✔	Inadequate separation of major-road movements.	Separate conflicting flows.	*Add (or lengthen) left-turn or right-turn bays.* *Increase right-turn radius.*
Delay	✔	✔	✔	--	--	--	Inadequate capacity for minor-road movements.	Increase approach capacity.	*Convert to roundabout.* *Convert to Yield control (if currently stop-controlled).* *Convert to traffic signal (with possible flash mode).* *Convert to multi-way stop control.*
	✔	✔	✔	--	--	--	Inadequate separation of minor-road movements.	Separate conflicting flows.	*Add a second lane on minor road.* *Increase right-turn radius.*
	✔	--	--	✔	--	--	Inadequate capacity for left-turn movements.	Increase capacity.	*Convert to traffic signal (with possible flash mode).* *Convert to roundabout.*
								Reduce demand.	*Prohibit left turns during peak hours with signing.* *Prohibit left turns permanently with channelization.*
Conflict[2]	✔	✔	✔	✔	--	--	Inadequate sight-distance along major road (possibly due to high major-road speed).	Remove view obstructions.	Relocate or remove objects blocking driver view. *Prohibit on-street parking permanently.* Offset opposing left-turn movements.
								Reduce needed sight distance.	Reduce speed limit (if justified). *Convert to multi-way stop control.* *Convert to traffic signal (with possible flash mode).*
	✔	✔	✔	✔	✔	✔	Inadequate protection for pedestrians.	Separate conflicting flows.	Add island channelization. Install crosswalks and/or sidewalks.
	✔	✔	✔	--	--	--	Inadequate sight-distance along minor road.	Remove view obstructions.	Relocate or remove objects blocking driver view. *Prohibit on-street parking permanently.*
								Change mode.	*Convert to two-way Stop control.* *Convert to roundabout.*
	✔	✔	✔	--	--	--	Lack of awareness of Yield sign (driver error).	Change mode.	*Convert to two-way Stop control.* *Convert to roundabout.*
								Provide guidance.	Add rumble strips. Add warning signs and/or devices.
	✔	✔	✔	--	--	--	Intentional stop violation.	Change mode.	*Convert to two-way Yield control.* *Convert to roundabout.*
	✔	✔	✔	--	--	--	Lack of awareness of Stop sign (driver error).	Change mode.	*Convert to roundabout.* *Convert to traffic signal (with possible flash mode).*
								Provide guidance.	Add rumble strips. Add warning signs and/or devices.

Notes:
1 - Movements: L = left-turn, R = right-turn, T = through movement.
2 - Conflicts can include erratic maneuvers, collisions, or non-compliance with control devices.
3 - Underline = alternative has an effect on traffic operation; *Italics* = alternative can be evaluated for viability in Step 3.

TABLE 2-3 Potential system-related, engineering improvement alternatives

Observed Problem	Approach [1]						Observed Cause	Corrective Strategy	Potential Alternatives [3]
	Minor			Major					
	L	T	R	L	T	R			
Delay and conflict[2]	✔	--	✔	✔	--	✔	Excessive volume due to traffic diversion.	Reduce diversion demand.	Add or improve alternative routes. Employ calming techniques to discourage diversion. Prohibit left-turns permanently with channelization.
Delay	✔	✔	✔	✔	--	--	Staggered platoons restrict capacity.	Modify arrival patterns.	Adjust signal timing at upstream signals. Relocate subject intersection (if possible).
								Concentrate and organize platoons.	*Convert to traffic signal (coordinated system).*
	✔	✔	--	✔	✔	✔	Queue spillback from downstream signal.	Increase downstream capacity.	Adjust signal timing at downstream signal. Modify signal coordination. Add traffic lanes at downstream signal.
								Separate conflicts.	Relocate subject intersection (if possible).
								Provide guidance.	Add advisory signing (i.e., do not block intersection).
	✔	✔	--	✔	✔	--	Queue spillback due to downstream unsignalized left turn.	Increase downstream capacity.	Add (or lengthen) bay at downstream intersection. Convert downstream intersection to traffic signal.
								Reduce demand.	Prohibit left-turns at downstream location.
								Provide guidance.	Add advisory signing (i.e., do not block intersection).

Notes:
1 - Movements: L = left-turn, R = right-turn, T = through movement.
2 - Conflicts can include erratic maneuvers, collisions, or non-compliance with control devices.
3 - Underline = alternative has an effect on traffic operation; *Italics* = alternative can be evaluated for viability in Step 3.

suitability on the basis of firsthand knowledge of site constraints and agency preferences. For example, if the site being studied is in a downtown area where right-of-way is limited, the analyst may choose to eliminate options that will require the acquisition of significant amounts of right-of-way. This screening is performed without the collection of additional data, so only options that would clearly not be acceptable should be eliminated.

Identify Alternatives Suitable for Immediate Implementation. All low-cost alternatives should be considered for immediate implementation and evaluation. An alternative's "cost" includes the direct cost of its implementation as well as any indirect cost to adjacent land users and the environment. Low-cost alternatives are defined as those alternatives that have a cost that is significantly less than that of the formal engineering study. Typical low-cost alternatives include advisory signing, revised pavement markings, and vegetation removal (to improve sight lines).

If any low-cost alternatives have been identified, the engineer can proceed directly to the implementation stage. In this stage, the low-cost alternative(s) would be programmed for implementation and monitored for effectiveness. If a follow-up observational study reveals that the alternatives implemented in this manner have not substantially improved the reported problems, then a new assessment process should be initiated.

Step 3. Select Viable Alternatives

The third step in the alternative identification and screening stage is to determine the viability of the candidate alternatives identified in Step 2. The tasks in making this determination are to

a. Gather information and
b. Assess and select alternatives

In the first task, relevant guidelines are identified, and corresponding data are collected to facilitate the evaluation of the candidate alternatives identified in Step 2. Then, in the second task, the guidelines are used to determine which of the candidate alternatives are more viable than the others.

The purpose of Step 3 is to screen out those alternatives that are not likely to have a significant positive effect on intersection operations. By screening out these alternatives, it is hoped that the list of alternatives to be evaluated during the engineering study (described in Chapter 3) will be reduced so that it includes only the most promising alternatives.

3-a. Gather Information

Overview. The procedure for selecting viable alternatives is based on a review of guidelines that indicate when an improvement alternative is likely to be effective. These guidelines are provided in the Guidelines section of this chapter. If desired, other guidelines may be added or substituted by the responsible agency. The Guidelines section addresses the following alternatives:

- Add flash mode to signal control,
- Convert to traffic signal control,
- Convert to multi-way stop control,
- Convert to two-way stop control,
- Convert to two-way yield control,
- Prohibit on-street parking,
- Prohibit left-turn movements,
- Convert to roundabout,
- Add a second lane on the minor road,
- Add a left-turn bay on the major road,
- Add a right-turn bay on the major road,
- Increase the length of the turn bay, and
- Increase right-turn radius.

(The order in which these alternatives are listed is arbitrary and is not intended to convey any sense of priority or importance.)

Alternative Categories. Some alternatives offered in Tables 2-2 and 2-3 are not included in the list above. These omissions occurred because (1) there is no formal guidance available in the literature, or (2) the alternative's effect cannot be quantified in terms of delay. Therefore, it is possible that only a subset of the candidate alternatives can be evaluated in this step.

Table 2-4 indicates the action to be taken based on the alternative's effect and guideline availability. At the onset of this task, Table 2-4 should be consulted for each candidate alternative to determine the appropriate action. Alternatives in Category I should be assessed using the process described in this section.

Identify Data. The next activity to be undertaken for this task is to gather the data needed to apply the guidelines applicable to each Category-I alternative. The specific types of data needed to evaluate each guideline are listed in Table 2-5. The list is generally complete for all of the guidelines shown; however, additional data may be needed in specific situations. In all cases, the analyst should review the guideline, as described in the Guidelines section, to determine if additional data are needed. Data needed for one guideline, Prohibit Left-Turn Movements, are not listed in the table because they are unique; however, they are described in the Guidelines section of this chapter.

Before gathering any data, the analyst should identify the specific data needed for the collective set of guidelines to be evaluated. Frequently, data needed for one guideline can be used for another guideline. The data collection plan should take advantage of such overlap to avoid redundancy.

If the traffic signal or multi-way stop control alternative is being considered, the analyst should note that only one component of the respective warrants or criteria need to be satisfied to designate the alternative as "viable." Thus, the analyst should determine which of the signal warrants and the multi-way stop control criteria will be evaluated and collect data only for these selected warrants and criteria. Often, the problems and causes identified in Step 2 will direct the selection of the warrants or criteria to be evaluated. When this is not the case, Hawkins and Carlson (*7*) recommend evaluation of *MUTCD 2000* Warrants 1, 2, and 3 as a first step, because the associated data require the least effort to collect.

Collect Data. The procedures for collecting the data needed to evaluate the selected guidelines will vary depending on whether the subject intersection exists or is proposed for construction. For existing intersections, appropriate data collection procedures are described in the *Manual of Transportation Engineering Studies* (*6*). For proposed intersections, techniques described in Appendix C can be used to estimate turn movement volumes from forecast average daily traffic demands. Regardless of the source, the data should represent traffic conditions occurring on an "average day" (i.e., a day

TABLE 2-4 Categories of candidate alternatives

Alternative Effect[1]	Guideline Availability[2]	Category
Operational	Yes, provided.	I. Viability can be assessed in Step 3.
	No, not provided.	II. Viability cannot be assessed, proceed to engineering study.
Safety	No, not provided.	III. Alternative assessment is beyond the scope of this guide.

Notes:
1 - Alternatives that have some direct effect on motorist delay are denoted by an underline in Tables 2-2 and 2-3.
2 - Refers to guidelines provided in the Guidelines section of this chapter.

TABLE 2-5 Data needed to evaluate guidelines

Data				Guideline[a, b, c] (numbers indicate hours of data needed)																	
Category	Approach		Smallest Interval	Signal Warrant								FM	MW		TW	Pk	Rb	2L	LB	RB	BL	RR
	Major	Minor		1	2	3	4	5	6	7	8		B	C								
Approach volume	✔		hour	8	4	1				8	8	24		8	1	8		1				
		✔	hour	8	4	1				8	8	24		8	1	8						
	✔	✔	day														✔					
Turn movement volume	✔		hour																1	1	1	1
		✔	hour															1			1	
	✔	✔	day														✔					
Heavy vehicle volume	✔	✔	hour																			1
	✔	✔	day														✔					
Pedestrian volume	✔		hour				4	1		4				8								
	✔	✔	day														✔					
Gap frequency	✔		hour				4	1														
Speed	✔		day	✔	✔	✔					✔			✔	✔				✔	✔	✔	✔
Progression quality	✔		day						✔													
Delay		✔	hour			1								1								
Intersection sight dist.	✔	✔	--														✔					
Safe approach speed		✔	--												✔							
Area population			--	✔	✔	✔					✔											
Number of lanes	✔	✔	--	✔	✔	✔				✔	✔	✔				✔						
Road classification	✔	✔	--												✔							✔
Bay length	✔	✔	--																		✔	
Right-turn radius	✔	✔	--																			✔
Crash history by type	✔	✔	1-year							✔			✔		✔							

Notes:

a - Guidelines:
FM - Flash mode signal control; MW - Multi-way stop control; TW - Two-way stop or yield control;
Pk - On-street parking restriction; Rb - Roundabout; 2L - Second lane on minor road;
LB - Left-turn bay; RB - Right-turn bay; BL - Bay length;
RR - Right-turn radius.

b - Numbers at the top of the Signal Warrant column and letters at the top of the MW column refer to the warrant or criterion in *MUTCD 2000* (*1*).

c- Numbers shown in the table indicate the minimum number of hours for which data are collected. These hours must represent the *highest* volume hours.

representing traffic volumes normally and repeatedly found at a location).

3-b. Assess and Select Alternatives

Overview. During this task, the Category-I alternatives are evaluated in order to develop a subset list of viable alternatives. The information gathered in Task 3-a is used to evaluate each alternative using the appropriate guideline. The steps involved in the application of each guideline are listed in the Guidelines section. Those alternatives that satisfy their corresponding guideline should be considered "viable" and evaluated more formally during the engineering study (described in Chapter 3).

At this point, some alternatives may appear less promising than others do in terms of their ability to solve a problem, and the analyst may be tempted to drop the less promising alternatives. However, these alternatives may provide the best balance between implementation impact and improvement effectiveness. The most appropriate alternative can only be accurately identified *after* the effectiveness of each alternative has been evaluated during the engineering study. In short, all viable alternatives should be advanced to the engineering study stage.

Traffic Signal Alternative Issues. Two issues should be addressed when the traffic signal alternative is found to be viable (i.e., one or more warrants are satisfied). Consideration of these issues is important because of the potential for the signal to affect intersection operation or safety negatively. The first issue relates to the belief of some engineers that the traffic signal is the most direct means of solving all intersection problems. The second issue relates to the presence of atypical (or problematic) conditions that may reduce the utility of the warrant check.

With regard to the first issue, the analyst might be inclined to drop all other alternatives when traffic signal control is found to be viable. However, the consequences of dropping the other alternatives can be significant. For example, studies show that when stop control is converted to signal control and the intersection volume "just" satisfies a warrant, the resulting overall delay often increases. Specifically, Williams and Ardekani (*8*) found that delays increased by as much as 113 percent; Kay et al. (*9*) found that delays increased 200 percent; and Bissell and Neudorff (*10*) found that delays increased by 10 seconds per vehicle (s/veh) when traffic signals were used to replace two-way stop control. Therefore, when the signal control alternative is found to be viable, the analyst is strongly encouraged to identify and advance other alternatives to the engineering study stage.

Regarding the second issue, atypical or unusual traffic, signalization, or geometric conditions may reduce the accuracy of the results of the warrant check. Some of the more frequently encountered problematic conditions include the following:

- Right-turn volume on the minor road,
- Heavy vehicles on the minor road,
- Pedestrian volumes,
- Progressive traffic flow on the major road,
- Three-leg intersection,
- Added through lane on the minor road,
- Left-turn bay,
- Right-turn bay, and
- Wide median on the major road.

(The order in which the conditions are listed is arbitrary and is not intended to convey any sense of priority or importance.)

With regard to the second issue, guidance provided in the Guidelines section can be used to determine if a problematic condition is likely to affect the results of the warrant check. If a problematic condition exists, then the effect of the condition on intersection operations should be carefully evaluated in the engineering study stage.

GUIDELINES

This section provides guidelines that can be used during the alternative identification and screening stage. These guidelines are presented in two separate sections with the following titles:

1. Guidelines for Use of Selected Geometric and Traffic Control Alternatives.
2. Conditions Affecting the Accuracy of Conclusions from the Signal Warrant Check.

The first section describes guidelines that can be used to evaluate the viability of 13 alternatives. The second section describes nine problematic traffic, signalization, or geometric conditions that reduce the accuracy of the results of the warrant check.

Guidelines for Use of Selected Geometric and Traffic Control Alternatives

Overview

This section provides guidelines that can be used to determine when various traffic control devices and geometric elements may be helpful in improving intersection operations or safety. These guidelines were obtained primarily from reference documents that are generally recognized as authoritative guides on engineering practice. Other sources for the guidelines include reports documenting significant research efforts whose recommendations are intended for nationwide application. The alternatives for which guidelines are provided in this section include

- Add flash mode to signal control,
- Convert to traffic signal control,
- Convert to multi-way stop control,
- Convert to two-way stop or yield control,
- Prohibit on-street parking,
- Prohibit left-turn movements,
- Convert to roundabout,
- Add a second lane on the minor road,
- Add a left-turn bay on the major road,
- Add a right-turn bay on the major road,
- Increase the length of the turn bay, and
- Increase the right-turn radius.

(The order in which the alternatives are listed is arbitrary and is not intended to convey any sense of priority or importance.)

The guidelines described in this section tend to be conservative such that they indicate only when an alternative *may* be helpful (i.e., viable). If an alternative is found to satisfy the guideline threshold conditions, then the effectiveness of the alternative should be verified through the conduct of an engineering study (as described in Chapter 3). The results of the engineering study should form the basis for any recommendation to implement an alternative. In contrast, if the guideline is not satisfied, then it should be assumed that the corresponding alternative is not viable and should be dropped from further consideration.

Finally, the guidelines provided in this section do not include all possible improvement alternatives. The guidelines provided are believed to correspond to the more commonly implemented alternatives. Judgment may be needed to identify viable alternatives for intersections that have unique operating or geometric conditions or when several alternatives are proposed for use in combination at a specific intersection.

Add Flash Mode to Signal Control

Introduction. Intersections with light-to-moderate traffic demands may benefit from traffic signal control during the hours of peak demand but may not derive benefit during off-peak hours. When this benefit results from a reduction in motorist delay, it may be useful to operate the signal in a flash mode during the off-peak hours. Flash mode may consist of a yellow/red combination or a red/red combination. For the yellow/red combination, the major road is flashed yellow and the minor road is flashed red.

Pusey and Butzer (*11*) indicate that flash mode operation has several benefits. These benefits can include

- Reduced stops and delays to major-road traffic,
- Reduced delay to minor-road traffic,
- Reduced electrical consumption, and
- Reduced vehicular fuel consumption and traffic noise.

A review of the literature by Kacir et al. (*12*) revealed that yellow/red flash mode may be associated with higher crash rates, especially when the ratio of major-road-to-minor-road volume is less than 2.0.

Guidance. Benioff et al. (*13*) define conditions in which flash mode is not likely to cause safety problems. Specifically, they recommend using flashing yellow/red when (1) the total major-road volume is less than 200 vehicles per hour (veh/h), or (2) when the ratio of major-road-to-minor-road volume is greater than 3.0. This recommendation is illustrated in Figure 2-2 as the unshaded area.

Kacir et al. (*12*) compared the delays produced by flashing yellow/red with those produced by traffic signal control. They found that delays were significantly reduced when (1) the major-road-to-minor-road volume ratio is greater than 3.0, (2) the major-road volume is less than 250 vehicles per hour per lane (veh/h/ln), and (3) the higher approach minor-road volume is less than 85 veh/h/ln. These recommendations were incorporated in Figure 2-2 with dashed lines (they are based on an assumed directional distribution of 55/45 percent for both roadways).

Application. The guidelines stated in the preceding section (and shown in Figure 2-2) are intended to minimize the operational impact of a traffic signal and maintain a reasonable degree of safety. This guideline assumes that traffic signal control is a viable alternative (i.e., that one or more signal warrants have been satisfied). Flash mode should be considered during each hour of the average day for which the major and minor volumes fall in the unshaded region. If flash mode is used, it should be used during extended periods (i.e., several hours per period) each day to minimize driver confusion.

Application of this guideline requires two types of data:

1. Major-road and minor-road approach volumes for each hour of the average day and
2. Major-road and minor-road approach through-lane count.

These traffic demands can be measured, or they can be estimated as a fraction of the average daily traffic demand. Kacir et al. (*12*) suggest the values listed in Table 2-6 can be used to estimate demands during the late-night hours. Also, the stopped drivers' unobstructed view of approaching unstopped vehicles should be verified before flash mode is implemented.

Convert to Traffic Signal Control

Introduction. An intersection with a properly designed and operated traffic control signal will have one or more of the following benefits (relative to an intersection without a traffic signal):

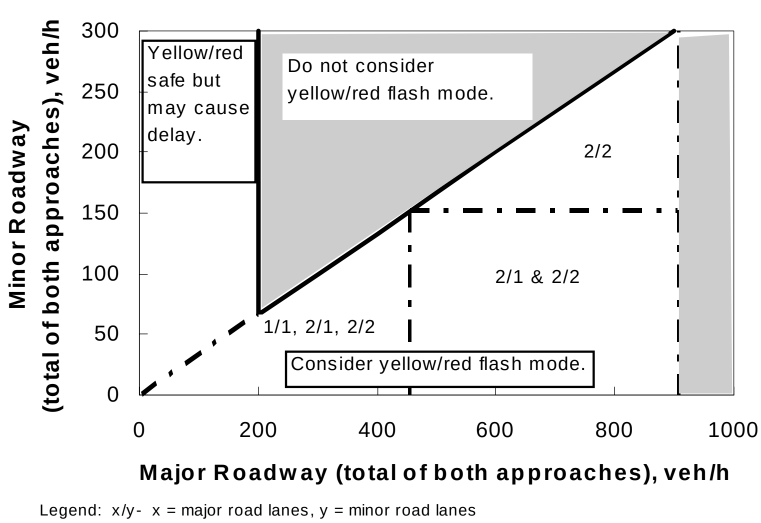


Figure 2-2. Guideline for use of signal flash mode.

TABLE 2-6 Hourly volume levels expressed as a percentage of average daily traffic demand

Volume Level	Hour of the Day					
	12 to 1 a.m.	**1 to 2 a.m.**	**2 to 3 a.m.**	**3 to 4 a.m.**	**4 to 5 a.m.**	**5 to 6 a.m.**
Percent of daily demand	0.8	0.5	0.4	0.2	0.2	0.5

- More orderly movement of traffic,
- Increased intersection capacity,
- Reduced frequency of certain types of collisions (e.g., right-angle),
- Continuous or nearly continuous movement of traffic along the through route, and
- Reduced delay to minor vehicular and pedestrian movements by interrupting heavy traffic at periodic intervals.

If the traffic signal is not properly designed or operated, one or more of the following problems may result:

1. Excessive delay to all traffic movements,
2. Excessive disobedience of the signal indication,
3. Increased frequency of diversion through neighborhoods, and
4. Increased frequency of certain types of collisions (e.g., rear-end).

These four problems may also be evident at a signalized intersection whose traffic signal is no longer needed.

Guidance. The *MUTCD 2000* describes eight warrants that define conditions in which a traffic control signal is likely to improve intersection safety, operations, or both. These warrants are listed in Table 2-7. If a warrant is met, then signal control may be appropriate. However, the appropriateness of the signal should be confirmed through the conduct of an engineering study (as described in Chapter 3).

Application. Collectively, the warrants listed in Table 2-7 address a wide range of factors that can affect safety and operations at an intersection. These factors include traffic volumes, volume variations during the day, approach geometry, major-road speed, crash frequency, motorist delay, and gap frequency in the major-road traffic stream. Often, only a subset of these warrants are evaluated because only one or two warrants are likely to be sensitive to the problem being experienced at the subject intersection.

Once the warrants to be evaluated are identified, the necessary data are collected using appropriate field study techniques or estimation methods. The data needed to evaluate each warrant are listed in Table 2-8. Procedures for collecting these data are described in the *Manual of Transportation Engineering Studies* (*6*). Data collection procedures are also described in "Traffic Signal Warrants: Guidelines for Conducting a Traffic Signal Warrant Analysis" (*7*). For proposed intersections, techniques described in Appendix C can be used to estimate turn movement volumes from forecast traffic demands.

Convert to Multi-Way Stop Control

Introduction. Multi-way stop control is most useful when intersection traffic volume is high enough to create frequent conflicts and the traffic volume is evenly split between the intersecting roads. Multi-way stop control may reduce the number of crashes more than will other, less-restrictive forms of control. Multi-way stop control can also result in less delay than other types of control when approach demands are nearly balanced and do not satisfy one or more of the *MUTCD 2000* Warrants 1, 2, or 3.

Guidance. The *MUTCD 2000* (*1, p. 2B-10*) describes four criteria that define conditions in which a multi-way stop is likely to improve intersection safety, operations, or both. If one

TABLE 2-7 Signal warrants in the *MUTCD 2000*

No.	Title	Basis
1	Eight-Hour Vehicular Volume	Sustained moderate volume overall or heavy volume major
2	Four-Hour Vehicular Volume	Heavy volume entering intersection during peak periods
3	Peak Hour	Very large volume or delay during the peak hour
4	Pedestrian Volume	Excessive delay to pedestrians and a large volume of pedestrians
5	School Crossing	Excessive delay to school children
6	Coordinated Signal System	Achieve/maintain progressive movement of traffic
7	Crash Experience	Frequent crashes and moderate volume
8	Roadway Network	Concentrate flow at intersection of two major roads

TABLE 2-8 Data needed to evaluate the *MUTCD 2000* signal warrants

Category	Approach		Smallest Interval	Notes [c]	Study [b]	Signal Warrant [a]							
	Major	Minor				1	2	3	4	5	6	7	8
Approach volume	✔		hour	*i, ii*	2	8	4	1				8	8
		✔	hour	*i, ii*		8	4	1				8	8
Pedestrian volume	✔		hour	--	13				4	1		4	
Gap frequency	✔		hour	--					4	1			
Speed	✔		day	*iii*	3	✔	✔	✔					✔
Progression quality	✔		day	--	4						✔		
Delay		✔	hour	--	5			1					
Area population			--	*iv*	--	✔	✔	✔					✔
Number of lanes	✔	✔	--	*v*	--	✔	✔	✔				✔	✔
Crash history by type	✔	✔	1-year	--	11							✔	

Notes:

a - Numbers at the top of each column indicate the warrant number. Numbers shown in the table indicate the minimum number of hours for which data are collected. These hours must represent the *highest* volume hours.

b - Numbers shown refer to the chapter in the *Manual of Transportation Engineering Studies* (*6*) that describes appropriate data collection procedures.

c - Supplemental notes:

i - Turn-movement volumes may be needed to determine the "effective" number of through lanes (see Note *v*).

ii - Signal Warrant 8 considers both existing and 5-year projected volumes during the average weekday and weekend day.

iii - Speed can be the posted, statutory, or measured 85th percentile speed.

iv - Only needed if the intersection lies in the built-up area of an isolated community.

v - Number of effective through lanes, as determined using guidance in Section 4C.01 of the *MUTCD 2000* (*1*).

criterion is met, multi-way stop control may be appropriate. However, this finding should be confirmed through the conduct of an engineering study (as described in Chapter 3). The four guidance criteria are as follows:

A. Where traffic control signals are justified, the multi-way stop is an interim measure that can be installed quickly to control traffic while arrangements are being made for the installation of the traffic control signal.
B. A crash problem, as indicated by five or more reported crashes in a 12-month period that are susceptible to correction by a multi-way stop installation. Such crashes include right- and left-turn collisions as well as right-angle collisions.
C. Minimum volumes:
   1. The vehicular volume entering the intersection from the major-street approaches (total of both approaches) averages at least 300 vehicles per hour for any 8 hours of an average day, and
   2. The combined vehicular, pedestrian, and bicycle volume entering the intersection from the minor street approaches (total of both approaches) averages at least 200 units per hour for the same 8 hours, with an average delay to minor-street vehicular traffic of at least 30 seconds per vehicle during the highest hour, but
   3. If the 85th percentile approach speed of the major-street traffic exceeds 65 km/h (40 mph), the minimum vehicular volume warrants are 70 percent of the above values.
D. Where no single criterion is satisfied, but where Criteria B, C.1, and C.2 are all satisfied to 80 percent of the minimum values. Criterion C.3 is excluded from this condition.

Option:

Other criteria that may be considered in an engineering study include:

A. The need to control left-turn conflicts.
B. The need to control vehicle/pedestrian conflicts near locations that generate high pedestrian volumes.
C. Locations where a road user, after stopping, cannot see conflicting traffic and is not able to safely negotiate the intersection unless conflicting cross traffic is also required to stop.
D. An intersection of two residential neighborhood collector (through) streets of similar design and operating characteristics where multi-way stop control would improve traffic operational characteristics of the intersection.

Application. Collectively, the criteria listed in the preceding section address a wide range of factors that can affect safety and operations at an intersection. These factors include traffic volume, volume variations during the day, approach geometry, major-road speed, crash frequency, and motorist delay.

Once the criteria to be evaluated are identified, the necessary data are collected using appropriate field study techniques or estimation methods. The data needed for each criterion are listed in Table 2-9. Procedures for collecting these data are described in the *Manual of Transportation Engineering Studies* (*6*).

Convert to Two-Way Stop or Yield Control

Introduction. The Stop sign is intended for intersection approaches on which drivers need to stop before proceeding

TABLE 2-9 Data needed to evaluate the *MUTCD 2000* multi-way stop control criteria

Category	Approach		Smallest Interval	Notes [c]	Study [b]	Guidance Criterion [a]	
	Major	Minor				B	C
Approach volume	✔		hour	--	2		8
		✔	hour	--			8
Pedestrian volume	✔		hour	--	13		8
Speed	✔		day	*i*	3		✔
Delay		✔	hour	--	5		1
Crash history by type	✔	✔	1-year	--	11	✔	

Notes:

a - Numbers shown indicate the minimum number of hours for which data are collected. These hours must represent the *highest* volume hours.

b - Numbers shown refer to the chapter in the *Manual of Transportation Engineering Studies* (*6*) that describes appropriate data collection procedures.

c - Supplemental Notes:

i- Speed can be the posted, statutory, or measured 85th percentile speed.

into the intersection. A benefit of stop control (over no control) is that it can improve overall intersection safety by clearly defining which movements have to yield the right-of-way. On the other hand, stop control has some disadvantages that include the following:

- Increased frequency of some collisions (e.g., rear-end collision),
- Increased road-user costs through increased fuel consumption and delay, and
- Increased air and noise pollution.

The Yield sign is used to inform drivers approaching an intersection that they do not have priority at the intersection but that stopping is not necessary if they can verify that the intersection will be clear when they reach it. Yield control is less restrictive than stop control, but more restrictive than "no control." Yield control tends to be associated with more collisions than stop control, but has a significantly lower road-user cost.

Guidance. The *MUTCD 2000* (*1, p. 2B-8*) presents four conditions where stop control may be appropriate:

A. Intersection of a less important road with a main road where application of the normal right-of-way rule would not be expected to provide reasonably safe operation.
B. Street entering a through highway or street.
C. Unsignalized intersection in a signalized area.
D. High speeds, restricted view, or crash records indicates a need for control by the STOP sign.

The *MUTCD 2000* (*1, 2B-12*) also offers several conditions where yield control may be appropriate. The conditions applicable to intersections are as follows:

A. When the ability to see all potentially conflicting traffic is sufficient to allow a road user traveling at the posted speed, the 85th percentile speed, or the statutory speed to pass through the intersection or to stop in a safe manner.
. . . . . .
C. At the second crossroad of a divided highway, where the median width is 9 m (30 ft) or greater. A STOP sign may be installed at the entrance to the first roadway of a divided highway and a YIELD sign may be installed at the entrance to the second roadway.
D. At an intersection where a special problem exists and where engineering judgment indicates the problem to be susceptible to correction by the use of the Yield sign.

With regard to the second Condition "A" above (i.e., "When the ability. . . ."), the minor-road driver's view of the major road should not be obstructed by curvature, grade, or objects in one or both of the quadrants adjacent to the minor-road approach. Guidelines for determining the minor-road driver's sight distance needs are described in Chapter 5 of the *Manual of Transportation Engineering Studies* (*6*).

Box (*14*) developed guidelines for use of traffic control signs at low-volume urban intersections. He recommended consideration of roadway classification, crash history, and the safe approach speed in determining the most appropriate control mode. The recommendations made by Box have been incorporated in Table 2-10.

Box (*14*) indicates that Table 2-10 should only be used for intersections with a total entering traffic volume of 300 veh/h or less during the peak hour. He also cautions that the no-control or yield-control options may not work well when the total entering volume exceeds 100 veh/h.

Application. The guidance stated in the preceding section indicates the conditions suitable to the use of two-way stop or yield control. Using this guidance requires five types of data:

1. Major- and minor-road approach volumes for the peak hour of the average day;
2. Major-road 85th percentile speed (posted speed can be substituted if data are unavailable);
3. Minor-road safe approach speed (based on available intersection sight distance);

TABLE 2-10 Candidate control for the minor-road approach[1]

Roadway Classification		Crash History[2] (1-yr / 3-yr)	Minor Road Control			
			Safe Approach Speed, km/h (mph)[3]:			
Major	Minor		< 15 (< 10)	15 to 30 (10 to 20)	31 to 50 (21 to 30)	≥ 50 (≥ 30)
Local	Local	< 2/4	Stop	Stop	Yield	none[4]
		≥ 2/4	Stop	Stop	Yield or Stop	Yield
Collector	Local	< 2/4	Stop	Stop	Yield	Yield
		≥ 2/4	Stop	Stop	Stop	Yield
Collector	Collector	< 2/4	Stop	Stop	Stop	Yield or Stop
		≥ 2/4	Stop	Stop	Stop	Yield or Stop

Notes:
1 - Table is only applicable to intersections in urban areas with a total entering volume of 300 veh/h or less during the peak hour. Two-way stop, multi-way stop, or signal control should be considered for higher volumes.
2 - Collisions susceptible to correction by stop or yield control (e.g., right-turn, left-turn, and right-angle collisions) on the lower-volume approach. Two collisions in a 12-month period or four in a 3-year period.
3 - Safe approach speed for minor-road drivers; based on an evaluation of their sight distance to major-road vehicles.
4 - "none": no control at intersection. May be limited to a total entering volume of 100 veh/h during the peak hour.

4. Major- and minor-road classification; and
5. Crash history for the previous 12-months as a minimum, but preferably for the previous 3 years.

These data would be used with Table 2-10 to determine the most appropriate minor-road control mode. The conditions from the *MUTCD 2000* listed (see the preceding Guidance section) should be assessed in combination with Table 2-10. If any of these conditions appear satisfied or if the conclusion reached from the use of Table 2-10 indicates that stop or yield control is appropriate, then stop or yield control may be a viable alternative for controlling the minor-road intersection approach.

Prohibit On-Street Parking

Introduction. Parking maneuvers into or out of on-street parking stalls can affect the operation and safety of the through traffic lane adversely. The stalls can be either angle or parallel with the curb. The time required for the parking maneuver is shorter for the angled stall than for a parallel parking stall. However, when leaving the stall, it is faster to leave a parallel parking stall than to leave an angled stall. Measurements indicate that the total blockage for both maneuvers is about 36 s. Chapter 16 of the *Highway Capacity Manual* (*15*) indicates that such maneuvers momentarily block the adjacent through lane and can significantly reduce intersection capacity if they occur within 76 m (250 ft) of the stop line.

Parking on the intersection approach can also have safety consequences. Cleveland et al. (*16*) report two studies indicating that the removal of parking can reduce crash frequency by 16 to 32 percent. Parked vehicles along one road can also create safety problems for drivers on the intersecting road by blocking the driver's view of conflicting traffic.

Guidance. *Special Report 125: Parking Principles* (*17*) describes guidelines for determining when to prohibit on-street parking. These guidelines indicate maximum flow rates that can be associated with on-street parking. Table 2-11 summarizes the guidance provided.

Application. The guidance described in the preceding section describes conditions that may justify the prohibition of on-street parking. Use of this guidance requires two types of data:

1. Major- and minor-road approach volumes for 8 or more hours of the average day.
2. Major- and minor-road approach through-lane count.

Table 2-11 should be consulted once for each hour of interest on a given intersection approach. Each approach is individually evaluated. If the combination of volume and lanes

TABLE 2-11 Guidelines for determining when to prohibit on-street parking

Type of Prohibition	Maximum Volume (1-direction), veh/h/ln	
	1-Lane Approach	2-or-more-Lane App.
Prohibit parking along entire street.	400	600
Prohibit parking within 46 m (150 ft) of intersection stop line.	300	500

exceeds the maximum value listed in the table, then parking prohibition should be considered a viable alternative for the subject approach during the specified hour. If it is determined that parking should be prohibited during any 1 hr, it would be preferable to prohibit parking during several hours (e.g., 7:00 a.m. to 6:00 p.m.) to minimize enforcement problems. However, parking prohibition during just the peak traffic demand hours can be considered.

Prohibit Left-Turn Movements

Introduction. Left-turning vehicles at an unsignalized intersection can create numerous operational and safety problems. The left-turn maneuver tends to require a longer service time than the through maneuver. In fact, even modest left-turn volumes can cause safety or operational problems, especially when there is inadequate storage for left-turn vehicles. When the right-of-way needed to provide this storage is not available, left-turn restriction (through regulation or channelization) is a means of eliminating these problems. However, the potential benefits of turn restriction should be carefully weighed against the increased travel time and trip length that is likely to be incurred by redirected motorists.

Guidance. Turn restrictions at an intersection should be carefully considered because they can cause traffic to divert to other, local roads. In general, the analyst should ensure that turn restriction is part of a larger program intended to identify the ultimate cause of the left-turn problem. Causes of such problems may include inadequate left-turn capacity at adjacent signalized intersections, inadequate left-turn storage at the subject intersection, and inadequate arterial travel speed (such that traffic diverts through neighborhoods via a left-turn).

The following guidelines can be used to identify the conditions suitable for left-turn restriction at existing intersections. These guidelines are based on the criteria offered by Koepke and Levinson (*18*).

- Left-turn-related delay, conflicts, or crash frequency should be at unacceptable levels.
- An alternative route is available for the redirected left-turn vehicles.
- The alternative route is not expected to add more than a few minutes to the redirected motorist's travel time.
- The intersection is in an urban or suburban area. (Note: in suburban settings, turn restriction is generally not found except where such treatments are part of an areawide circulation plan.)

If operational problems are only experienced during the peak hours, then left-turn restriction during these hours may be a viable alternative. If operational problems exist throughout the day and provision of a left-turn bay (of adequate length) is not an option, then full-time left-turn restriction may be a viable alternative. Regardless of the duration of the restriction, all four of the above criteria should be satisfied before turn restriction is given further consideration.

Application. The guidance described in the preceding section can be used to determine if left-turn restriction might improve operations or safety at an intersection. Evaluation of this guidance requires three types of data (as measured during the morning peak hour, afternoon peak hour, and one representative off-peak hour of the average day):

1. Delay resulting from left-turn vehicles queued in a through lane because of nonexistent or inadequate bay storage;
2. Left-turn-related traffic conflicts (or left-turn-related crash history for the previous year); and
3. Travel time for the likely alternative route(s).

The left-turn-related delay, conflict, and crash data should be used to determine if turn restriction is needed and, if needed, for what hours of the day. When turn restrictions only need to be in place during certain times of the day, the turn restriction should be shown by (1) a variable message sign, (2) an internally illuminated sign whose legend is visible only during the hours where the prohibition is applicable, or (3) permanently mounted signs with a supplementary legend stating the hours when the prohibition is in effect. If turn restriction is needed throughout the day, then raised-curb channelization should be considered to maximize compliance.

Convert to Roundabout

Introduction. The modern roundabout can offer operational and safety benefits that exceed those offered by other forms of intersection control for certain conditions. Robinson et al. (*19*) indicate that roundabouts have the following advantages:

1. They are effective traffic-calming devices because they reduce speeds.
2. They can reduce the frequency and severity of some crashes (e.g., right-angle, head-on, and turn-related).
3. They result in less delay than multi-way stop control.
4. They result in less delay than two-way stop control when volumes are sufficient to cause operational problems at the two-way stop-controlled intersection.
5. They result in less delay than signal control when volumes do not exceed the roundabout's capacity.

The roundabout also has some characteristics that may preclude its use at some locations. These characteristics are as follows:

1. Roundabouts may require more right-of-way than a conventional intersection.

2. They may not be easily traversed by large or over-sized trucks.
3. Roundabouts may not yield efficient operation if the major-road volume greatly exceeds the minor-road volume.
4. They may not be conducive to through movement progression in coordinated signal networks.
5. They may not be conducive to serving pedestrian or bicycle traffic.

Guidance. The report prepared by Robinson et al. (*19*) provides some information on the conditions where modern roundabouts are well suited. These conditions have been restated in Table 2-12 in terms of questions to be answered when considering a roundabout. The questions have been worded so that the roundabout becomes a more viable alternative as the number of questions answered "Yes" increases.

The maximum daily service volume needed to answer Question 2 in Table 2-12 can be obtained from Figure 2-3. This figure was developed by Robinson et al. (*19*) and assumes that (1) the peak hour has 10 percent of the daily volume, (2) one direction of flow on each road has 58 percent of the total two-way flow, (3) 10 percent of each approach volume is turning right, and (4) the maximum service volume equates to 85 percent of capacity. Figure 2-3 applies to a four-leg roundabout; if applied to a three-leg roundabout, the values obtained from the figure should be multiplied by 0.75. An equation is offered in the footnote to Table 2-12 to facilitate the computation of maximum service volume.

Application. The guidance stated in the preceding section indicates the conditions in which a roundabout may be desirable. Application of this guidance requires four types of data:

1. Major- and minor-road approach volumes for the average day;
2. Major- and minor-road turn movement volumes for the average day (used to compute average left-turn percentage);

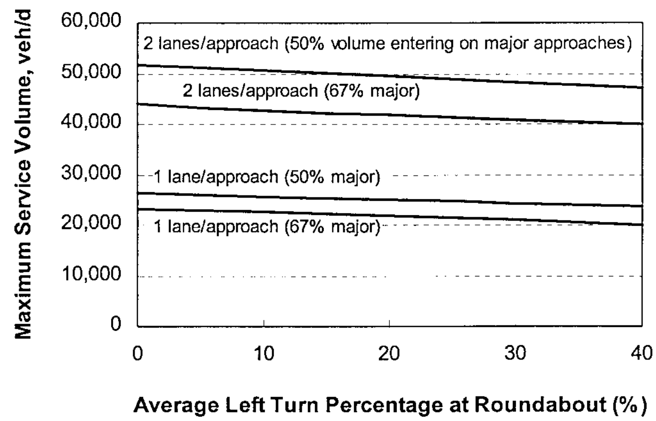


Figure 2-3. Maximum daily service volumes for a four-leg roundabout.

3. Major- and minor-road approach sight distance; and
4. Major- and minor-road pedestrian, bicycle, and heavy vehicle volumes for the average day.

Any evaluation of operational benefits derived as a result of the roundabout should consider operations *throughout* the day because the roundabout offers significant operational benefit (over stop or signal control) during off-peak hours.

Add a Second Lane on the Minor Road

Introduction. The quality of service provided to the minor-road movements at a two-way stop-controlled intersection is dependent on the number of lanes provided. Typically, one lane is shared by all movements on the minor-road approach; however, sometimes a turn bay or second through lane is added to reduce the delay to selected movements. A second lane can be added as a right-turn bay, a left-turn bay, or a second shared lane (i.e., through plus left in the inside lane and through plus right in the outside lane). Of these options, adding a right-turn bay is often the most effective as it can significantly reduce the delay to the right-turn movement; it

TABLE 2-12 Worksheet to determine the suitability of modern roundabout

Question	Y/N
1. Will operation as an uncontrolled or a two-way-stop-controlled intersection yield unacceptable delay?	
2. Is the daily entering volume less than the maximum daily service volume for a roundabout?[1]	
3. Is the subject junction located outside of a coordinated signal network?	
4. Is the ratio of major-road-to-minor-road volume less than 5.0?	
5. Is the entering driver's view free of sight obstructions (e.g., due to grade, curvature, or vegetation)?	
6. Will the subject junction be infrequently used by large or over-sized trucks?	
7. Will the subject junction be infrequently used by pedestrians and bicyclists?	

Note:
[1]Maximum service volume (4-legs), veh/d $= 3600 + 9000\,lanes\,(1 + \frac{81}{major}) - 94\,lefts$, where *lanes* = number of roundabout entry lanes per approach, *major* = percent of volume entering on the major-road approaches, and *lefts* = percent of left-turns at junction. Maximum service volume (3-legs) = 0.75 * maximum service volume (4-legs).

can also indirectly reduce the delay to the left-turn or through movements by lessening their need to compete for service with the right-turn movement.

One disadvantage of adding a lane to the minor-road approach is that it may require reallocating the existing pavement or widening of the approach cross section. Sometimes the pavement width needed for the additional lane is available within the existing roadway cross section. In this instance, the only impact is a reallocation of the paved surface through modification of the pavement markings. However, in downtown settings this reallocation may require the removal of some curb parking stalls and can affect adjacent business significantly. Occasionally, the cross section must be widened to provide for the additional lane. If the needed lane width can be provided within the available right-of-way, the cost may be limited to that of construction. However, if additional right-of-way is needed, the costs of acquiring this property in urban settings can be high.

Guidance. The literature does not offer guidance regarding conditions where a second approach lane would benefit from the operation of a minor-road approach. However, the procedures in Chapter 17 of the *Highway Capacity Manual 2000* (*15*) can be used to identify major- and minor- road volume combinations that would benefit operationally from the provision of a second approach lane or bay. Bonneson and Fontaine (*20*) developed Figure 2-4 using these procedures and an assumed upper limit of 0.7 for the shared-lane, minor-road volume-to-capacity ratio.

Application. Figure 2-4 indicates the conditions that may justify the use of two approach lanes. Use of the information in this figure requires two types of data:

1. Major-road approach volume for the peak hour of the average day and
2. Minor-road turn movement volume for the peak hour of the average day (used to compute right-turn percentage).

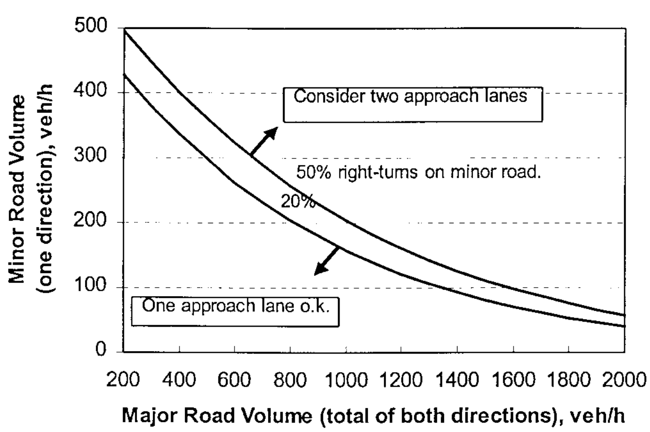


Figure 2-4. Guideline for determining minor-road approach geometry at two-way stop-controlled intersections.

Figure 2-4 would be used once for each minor-road approach to the intersection. The appropriate trend line would be identified on the basis of the percentage of right-turns on the subject minor-road approach. If the volume combination for the major and minor roads intersects above or to the right of this trend line, a second traffic lane should be considered for the subject minor-road approach. If a bay is selected for addition to the intersection, it should be long enough to store vehicles 95 percent of the time (i.e., the bay should not overflow more than 5 percent of the time). Techniques for estimating the 95th percentile storage length are provided in the section, Increase the Length of the Turn Bay.

Add a Left-Turn Bay on the Major Road

Introduction. Provision of a left-turn bay on the major road to a two-way stop-controlled intersection can significantly improve operations and safety at the intersection. A left-turn bay effectively separates those vehicles that are slowing or stopped to turn from those vehicles in through traffic lanes. This separation minimizes turn-related crashes and eliminates unnecessary delay to through vehicles. Data reported by Neuman (*21*) indicate that the crash rate for unsignalized intersections can be reduced by 35 to 75 percent through the provision of a left-turn bay.

One disadvantage of adding a bay to the major-road approach is that it may require reallocating the existing pavement or widening of the approach cross section. Sometimes the pavement width needed for the additional lane is available within the existing roadway cross section. However, in downtown settings this reallocation may require the removal of some curb parking stalls and can affect adjacent business significantly. Occasionally, the cross section must be widened to provide for the turn bay. If the needed width can be provided within the available right-of-way, the cost may be limited to that of construction. However, if additional right-of-way is needed, the costs of acquiring this property in urban settings can be high.

Guidance. Neuman (*21*) suggests that the following guidelines should be used to determine when to provide a left-turn bay on the major road of a two-way stop-controlled intersection:

1. A left-turn lane should be considered at any median crossover on a divided, high-speed road.
2. A left-turn lane should be provided on the unstopped approach of a high-speed rural highway when it intersects with other arterials or collectors.
3. A left-turn lane is recommended on the unstopped approach of any intersection when the combination of intersection volumes intersect above or to the right of the appropriate trend line shown in Figure 2-5.

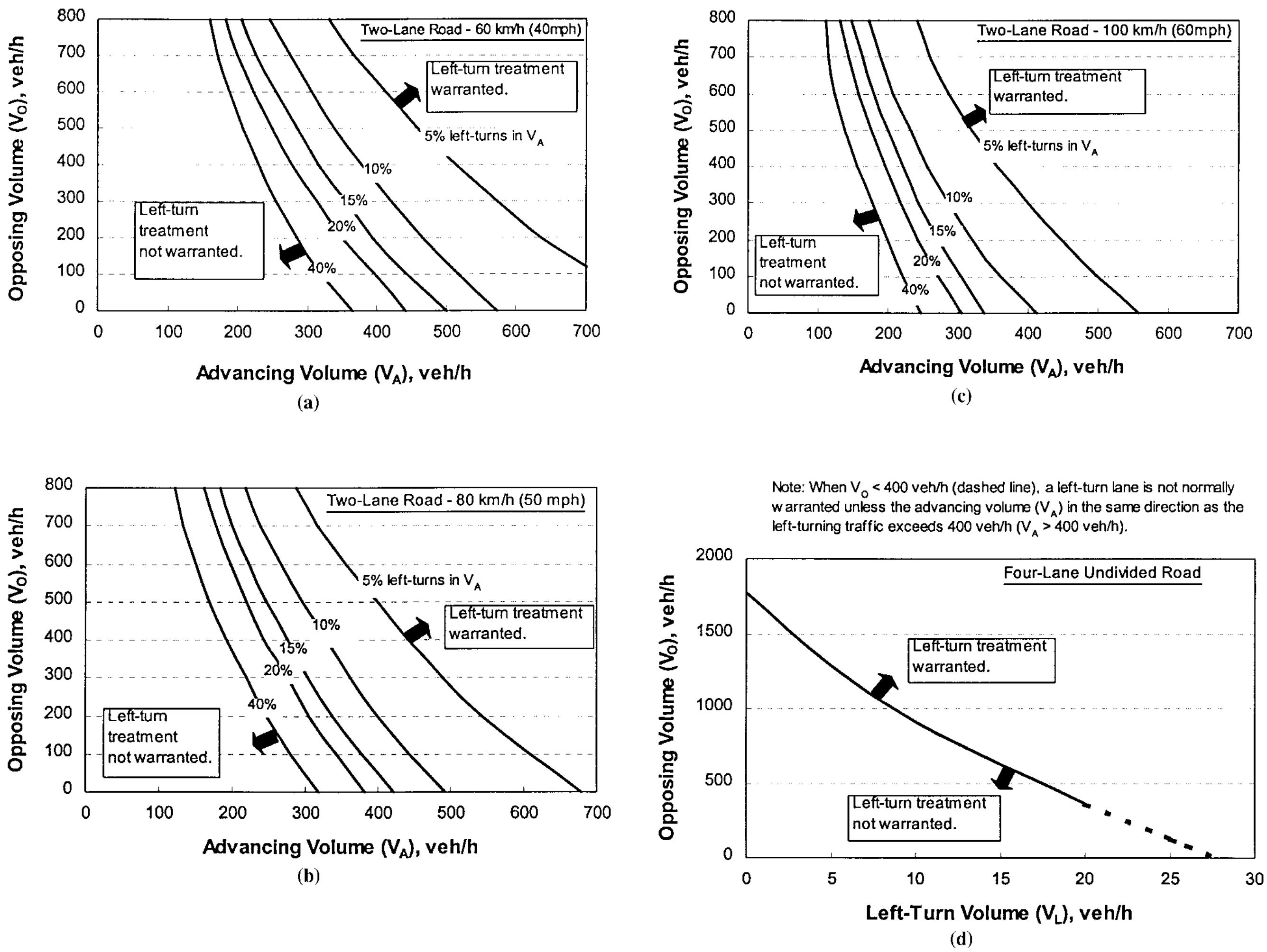


Figure 2-5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

Application. The guidance stated in the preceding section defines the conditions that may justify the provision of a left-turn bay. Application of this guidance requires two types of data:

1. Major-road turn movement volume for the peak hour of the average day and
2. Major-road 85th percentile speed (posted speed can be substituted if data are unavailable).

Use of Figure 2-5 requires determination of the opposing volume, the advancing volume, and the operating speed. The opposing volume should include only the right-turn and through movements on the approach across from (and heading in the opposite direction of) the subject major-road approach. The advancing volume should include the left-turn, right-turn, and through movements on the subject approach. The operating speed can be estimated as the 85th percentile speed. If the operating speed does not coincide with 60, 80, or 100 km/h (i.e., 40, 50, or 60 mph), then interpolation can be used or, as a more conservative approach, the operating speed can be rounded up to the nearest speed for which a figure is provided.

In application, Figure 2-5 is used once for each major-road approach to the intersection. The appropriate trend line is identified on the basis of the percentage of left-turns on the subject major-road approach. If the advancing and opposing volume combination intersects above or to the right of this trend line, a left-turn bay should be considered for the subject approach. If a bay is included at the intersection, it should be long enough to store left-turn vehicles 99.5 percent of the time (i.e., the bay should not overflow more than 0.5 percent of the time). Techniques for estimating this storage length are provided in the section, Increase the Length of the Turn Bay.

Add a Right-Turn Bay on the Major Road

Introduction. Provision of a right-turn bay on the major road to a two-way stop-controlled intersection can significantly improve operations and safety at the intersection. A right-turn bay effectively separates those vehicles that are slowing or stopped to turn from those vehicles in the through traffic lanes. This separation minimizes turn-related collisions (e.g., angle, rear-end, and same-direction-sideswipe) and eliminates unnecessary delay to through vehicles.

One disadvantage of adding a bay to the major-road approach is that it may require reallocating the existing pavement or widening of the approach cross section. Sometimes the pavement width needed for the additional lane is available within the existing roadway cross section. However, in downtown settings this reallocation may require the removal of some curb parking stalls and can affect adjacent business significantly. Occasionally, the cross section must be widened to provide for the turn bay. If the needed width can be provided within the available right-of-way, the cost may be limited to that of construction. However, if additional right-of-way is needed, the costs of acquiring this property in urban settings can be high.

Guidance. Hasan and Stokes (*22*) developed guidelines for determining when to provide a right-turn bay on the major road of a two-way stop-controlled intersection. These guidelines were based on an evaluation of the operating and collision costs associated with the right-turn maneuver relative to the cost of constructing a right-turn bay. The operating costs included those of road-user fuel and delay. Separate guidelines were developed for two-lane and four-lane roadways. These guidelines are shown in Figure 2-6.

Application. The guidance described in the preceding section defines conditions that may justify the provision of a right-turn bay. Application of this guidance requires two types of data:

1. Major-road turn movement volume for the peak hour of the average day and
2. Major-road 85th percentile speed (posted speed can be substituted if data are unavailable).

Figure 2-6 should be consulted once for each major-road approach. If the combination of major-road approach volume and right-turn volume intersects above or to the right of the trend line corresponding to the major-road operating speed, then a right-turn bay is a viable alternative.

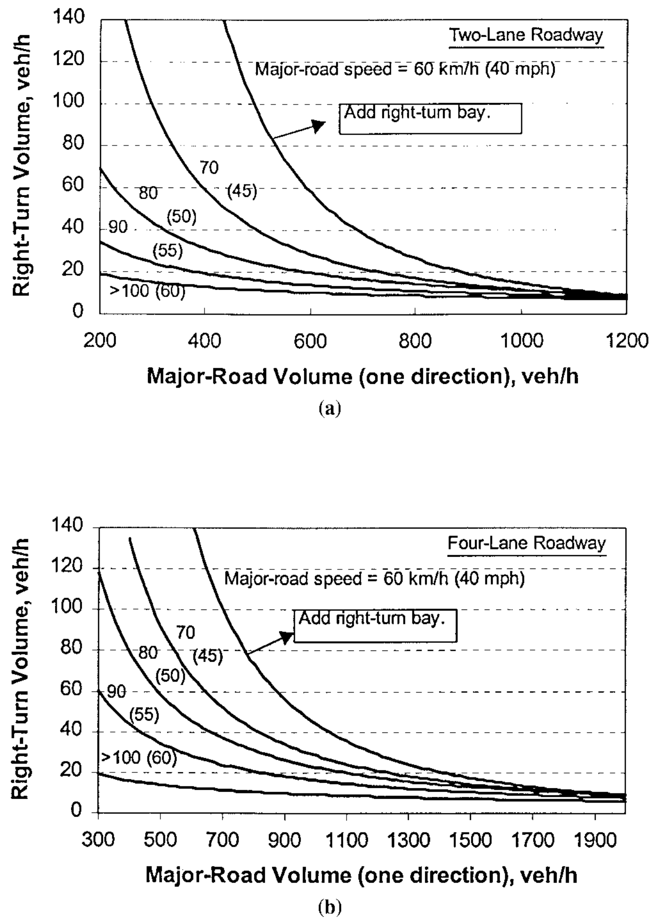


Figure 2-6. Guideline for determining the need for a major-road right-turn bay at a two-way stop-controlled intersection.

Increase Length of Turn Bay

Introduction. Turn bay length can affect the safety and operation of the intersection approach significantly. This effect becomes more negative as the frequency with which vehicles exceed the available storage increases. Also, for unstopped approaches, this effect becomes more negative as more of the turning vehicle's deceleration occurs in the through lane, prior to the bay. The need to provide adequate storage length, deceleration length, or both is dependent on the type of approach control used and whether the vehicle is turning left or right. Table 2-13 identifies the appropriate bay

TABLE 2-13 Turn-bay length components at unsignalized intersections

Approach Control	Length Components	
	Left-Turn Bay	Right-Turn Bay
Unstopped	Storage Length + Deceleration Length	Deceleration Length
Stopped	Storage Length	Storage Length

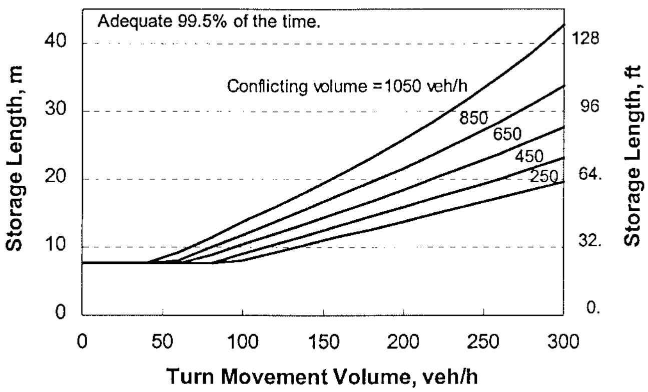


Figure 2-7. Guideline for determining if the bay storage length for an unstopped approach is adequate.

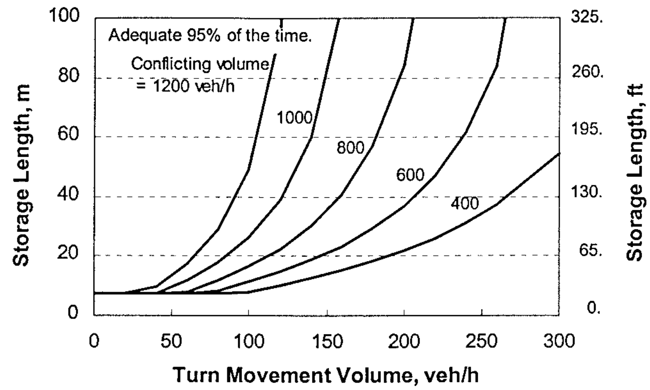


Figure 2-8. Guideline for determining if the bay storage length for a stopped approach is adequate.

length components for typical combinations of approach control and turn movement.

Guidance. A turn bay should be long enough to store waiting vehicles almost all of the time and, in doing so, prevent conflict between the through and turning movements. A bay equal in length to the 95th percentile queue (which allows for a 5-percent probability of overflow) is typically used for most stop- or signal-controlled approaches. A bay length corresponding to a 0.5-percent probability of overflow is recommended by Harmelink (*23*) for unstopped approaches in recognition of the greater speed differential between through and turning movements.

Figures 2-7 and 2-8 illustrate the combination of volume and storage length that limit overflows to 0.5 and 5 percent, respectively. The trends in these figures are based on the queue length equation reported by Harmelink (*23*). The capacity needed for this equation was estimated using the procedures in Chapter 17 of the *Highway Capacity Manual 2000* (*15*). The development of Figures 2-7 and 2-8 is described elsewhere by Bonneson and Fontaine (*20*). A practical minimum storage length of 8 m (25 ft) is reflected in both figures.

As indicated in Table 2-13, a bay on the unstopped approach to an intersection should also provide sufficient length for the turning vehicle to decelerate once it is clear of the adjacent through traffic lane. A report by Koepke (*24*) provides an indication of the bay length needed for turn vehicle deceleration. However, the lengths cited by Koepke include both deceleration length and bay taper length. Subtraction of the taper length [estimated at 35 m (120 ft)] from the reported lengths leaves a conservative estimate of the length of full-width turn bay needed to facilitate the deceleration process. This length is shown in Figure 2-9.

Application. The guidelines described in the preceding section can be used to determine when turn-bay length may need to be increased to minimize operational or safety problems associated with short turn bays. Application of this guideline requires three types of data:

1. Major- and minor-road turn movement volumes for the peak hour of the average day;
2. Major-road 85th percentile speed (posted speed can be substituted if data are unavailable); and
3. Major- and minor-road bay lengths (taper length should be excluded).

In application, each turn movement is considered separately. Table 2-13 is consulted first to determine if storage length, deceleration length, or both should be provided within the bay for the subject approach. Next, Figures 2-7, 2-8, and 2-9 are consulted. The subject movement volume and conflicting volumes should be used to determine the minimum length(s) needed from the appropriate figure(s). The operat-

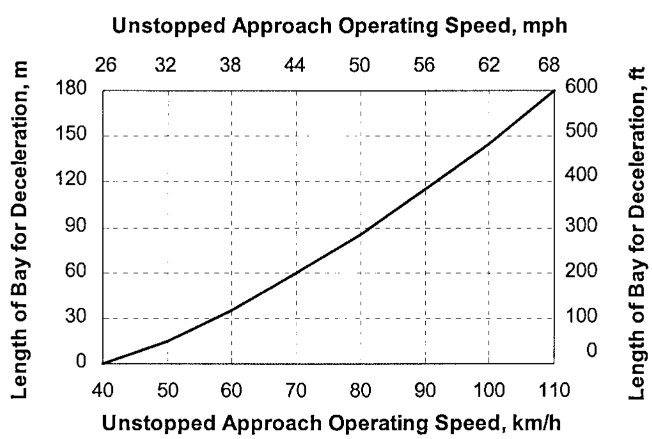


Figure 2-9. Guideline for determining if the deceleration length component of a bay is adequate.

ing speed can be estimated as the 85th percentile speed. The conflicting volume should reflect those traffic streams that have priority over the subject movement and that cross its travel path within the intersection [see Chapter 17 of the *Highway Capacity Manual 2000* (*15*) for a more detailed definition of conflicting volume]. If the available bay length is less than the computed minimum length, then an increase in bay length is a viable alternative.

Increase the Right-Turn Radius

Introduction. The radius of the right-turn path can affect the operation and safety of an intersection. Larger radii have the following advantages:

- On the major-road approach, they allow for higher speed turns and by that, reduce delay to following through vehicles.
- On the minor-road approach, they widen the throat of the approach and effectively act as a short right-turn bay.
- They allow large trucks and buses to turn without encroachment into adjacent lanes or onto adjacent property.

Larger radii also have disadvantages. These disadvantages include the following:

- They require more right-of-way.
- They may reduce pedestrian safety through increased crossing distance and higher turn speeds.

The second disadvantage noted for large radii can be minimized through the inclusion of island channelization on the outside of the right-turn path. This island would have a triangular shape and serve as a pedestrian refuge.

Guidance. The AASHTO document, *A Policy on Geometric Design of Highways and Streets* (*Green Book*) (*5*) provides general guidance on the minimum radii needed at intersections in suburban or urban areas. These guidelines are reproduced in Table 2-14.

Larger radii should be used when right-of-way is available and pedestrian volumes are minimal. If larger radii are desired and right-of-way is restricted, then a simple-curve-radius-with-taper or a 3-centered curve may be used to increase the "effective" radius without using as much right-of-way as a simple radius. Desirable geometrics for these curves are described in Tables IX-1 and IX-2 of the *Green Book* (*5*). If larger radii are desired and pedestrian volumes are significant, then island channelization should be provided. The *Green Book* lists several 3-centered curve designs in Table IX-4 that produce islands meeting the minimum island size requirements.

Hasan and Stokes (*22*) developed guidelines for determining when to provide a simple-curve-radius-with-taper design on the major road of a two-way stop-controlled intersection. These guidelines were based on an evaluation of the operating and collision costs associated with the right-turn maneuver relative to the cost of constructing a right-turn bay. The operating costs included those of road-user fuel and delay. Separate guidelines were developed for two-lane and four-lane roadways. These guidelines are shown in Figure 2-10.

Application. The guidance provided in the preceding section can be used to determine if an intersection has adequate right-turn radius design. Application of the *Green Book* (*5*) guidance requires three types of data:

1. Heavy vehicle volume during the peak hour of the average day;
2. Major- and minor-road classification; and
3. Major- and minor-road right-turn radius, measured to the edge of the traveled way.

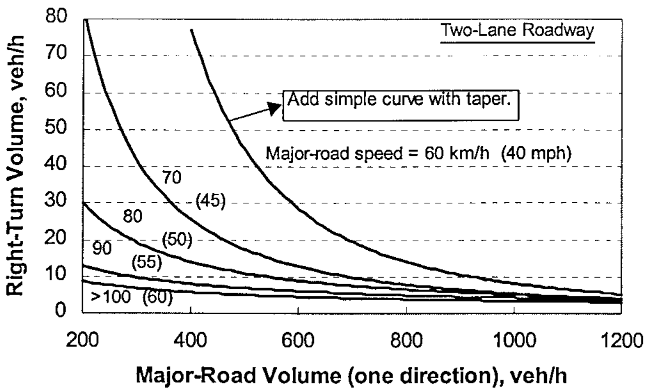
As a first step, the engineer should decide whether a simple curve radius or a more complicated curve design is appropriate for the subject intersection. In general, a simple curve radius is appropriate for low-speed approaches to urban intersections; otherwise, a simple-curve-radius-with-taper or a 3-centered curve design should be considered.

If a simple curve radius is deemed appropriate, Table 2-14 is consulted once for each intersection approach. If the existing right-turn radius on a given approach does not equal or exceed the value listed, then increasing the right-turn radius should be considered a viable alternative.

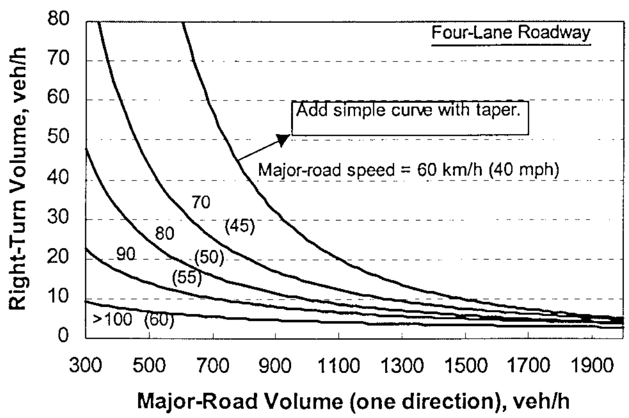
TABLE 2-14 Guideline for determining the need for a larger simple radius

	Local or Collector Street[1]		Arterial Street	
	Few Trucks[2]	Many Trucks	Few Trucks[2]	Many Trucks
Minimum Simple Radius[3], m (ft)	4.5 (15)	7.5 (25)	9 (30)	15 (50)

Note:
1 - If the local or collector street cross-section width is less than 11 m (36 ft), then use the "arterial street" criteria.
2 - "Few Trucks" is defined herein as being less than 10 trucks per hour.
3 - Radius is measured to the inside edge of the traveled way.



(a)



(b)

Figure 2-10. Guideline for determining the need for a simple-curve-radius-with-taper.

If a simple-curve-radius-with-taper or 3-centered curve is appropriate, the guidance developed by Hasan and Stokes (*22*) should be consulted. Use of this guidance requires two types of data:

- Major-road turn movement volume for the peak hour of the average day.
- Major-road 85th percentile speed (posted speed can be substituted if data are unavailable).

In application, Figure 2-10 is consulted once for each major-road approach. If the combination of major-road approach volume and right-turn volume intersects above or to the right of the trend line corresponding to the major-road operating speed (estimated as the 85th percentile speed), then a simple-curve-radius-with-taper or a 3-centered curve should be considered a viable alternative.

Conditions Affecting the Accuracy of Conclusions from the Signal Warrant Check

Overview

The *MUTCD 2000* (*1*) signal warrants identify threshold traffic volumes below which an intersection is not likely to be amenable to traffic signal control. When a check of the warrants reveals that volumes exceed these thresholds, then an engineering study should be conducted to verify whether the signal will improve intersection safety or operations. The study is needed because the traffic or geometric conditions at the subject intersection may differ enough from those assumed in the warrants that the results of the warrant check would be inaccurate.

Table 2-15 lists common problematic conditions. These conditions are described in this section. Included in this description is information that can be used to determine if the condition is likely to exist at the subject intersection. If it is determined that a problematic condition exists, then the effect of the condition on intersection operations should be fully evaluated during the engineering study stage.

Right-Turn Volume on the Minor Road

Minor-road right-turn vehicles at an unsignalized intersection incur less delay than the left-turn or through vehi-

TABLE 2-15 Problematic traffic, signalization, and geometric conditions

Category	Problematic Condition	Applicable Intersection Approach	
		Major Road	Minor Road
Traffic	1. Right-turn volume	--	✔
	2. Heavy vehicles	--	✔
	3. Pedestrian volumes	✔	✔
Signalization	4. Progressive traffic flow	✔	--
Geometry	5. Three-leg intersection	--	✔
	6. Added through lane	✔	--
	7. Left-turn bay	✔	✔
	8. Right-turn bay	--	✔
	9. Wide median	✔	--

Notes:
"--"- condition does not apply to the corresponding intersection approach.

cles. Thus, an intersection where minor-road drivers are primarily turning right is less likely to derive benefit from signalization than one where most drivers are crossing through or turning left.

The effect of minor-road right-turns is also recognized in Section 4C.01 of the *MUTCD 2000* (*1*). For shared-lane approaches, the *MUTCD 2000* indicates that a portion of the right-turning volume should be subtracted from the minor-road volume when evaluating the volume-based signal warrants. For minor-road approaches with an exclusive right-turn bay *and* adequate right-turn capacity, the *MUTCD 2000* indicates that all of the right-turn volume can be subtracted from the minor-road volume and that the right-turn bay should not be included in the count of approach lanes. Without these adjustments, it is likely that an intersection with moderate-to-high right-turn volume will yield inaccurate conclusions from the warrant check.

The following method can be used to determine if right-turn volume on the minor road could influence the warrant check. The method is based on signal warrant analysis guidelines developed by the Utah Department of Transportation (*25*). Using this method, the actual right-turn volume is reduced on the basis of consideration of the major-road volume that conflicts with the right-turn movement, the number of traffic lanes serving the conflicting volume, and the geometry of the subject minor-road approach. The relationship between these factors is illustrated in Figure 2-11.

To determine if a heavy right-turn volume might mislead the warrant check, a second warrant check can be performed using an adjusted minor-road volume. This adjusted volume is computed as follows:

$$\text{Adjusted Minor Road Volume} = \text{Larger of: } \begin{bmatrix} V_7 + V_8 + V_9 - V_{r9} \\ V_{10} + V_{11} + V_{12} - V_{r12} \end{bmatrix}$$

with,

$$V_{c9} = 0.5V_3 + \frac{V_2}{N_2}$$

$$V_{c12} = 0.5V_6 + \frac{V_5}{N_5}$$

where

V_i = volume for movement *i* (movement numbers are shown in Figure 2-12), veh/h;

V_{r9} = right-turn volume reduction for movement 9 (obtained from Figure 2-11 using conflicting major-road volume V_{c9}), veh/h;

V_{r12} = right-turn volume reduction for movement 12 (obtained from Figure 2-11 using conflicting major-read volume V_{c12}), veh/h;

V_{c9} = conflicting major-road volume for movement 9; veh/h;

V_{c12} = conflicting major-road volume for movement 12, veh/h; and

N_i = number of approach lanes serving through movement *i*.

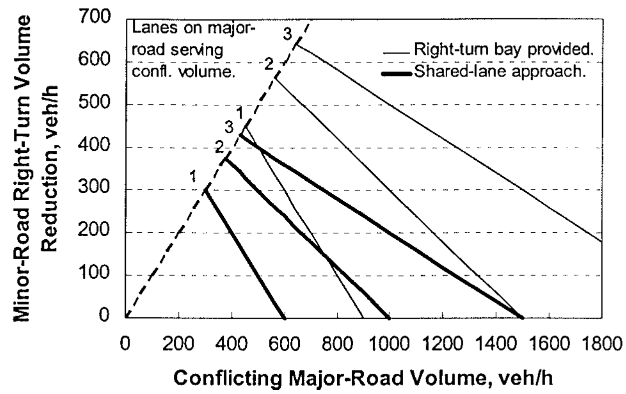


Figure 2-11. Minor-road right-turn volume reduction for warrant check.

The volume reduction cannot exceed the corresponding right-turn volume (i.e., $V_9 - V_{r9} \geq 0$ and $V_{12} - V_{r12} \geq 0$). Also, the "Right-turn bay provided" case in Figure 2-11 can be used for shared-lane approaches when the shared lane functions as a de facto right-turn lane.

If the second warrant check yields different conclusions than the first warrant check (i.e., the check based on unadjusted volumes), then right-turn volumes may be enough to affect the accuracy of the warrant check. In this situation, it is recommended that the effect of right-turns be fully examined during the engineering study.

Heavy Vehicles on the Minor Road

Heavy vehicles on the minor-road approach to an unsignalized intersection should logically increase overall delay. This effect would result from the relatively large gaps that the drivers of such vehicles need to cross or enter the major-road traffic stream safely. Thus, an intersection with a significant number of heavy vehicles is likely to derive more benefit from signalization than one where there are fewer heavy vehicles.

A study conducted by Henry and Calhoun (*26, p. 31*) revealed that the volume of heavy vehicles must be "unusually high" to have a significant impact on the warrant evaluation process. Their report indicates that the heavy-vehicle percentage would have to exceed 5 percent during the peak traffic hour to constitute "unusual" conditions. Therefore, if the percentage of heavy vehicles exceeds 5 percent or if the analyst is uncertain of the effect of heavy vehicles (perhaps when combined with other factors such as grade) on intersection operations, then the effect of heavy vehicles should be fully evaluated during the engineering study.

Pedestrian Volume

Heavy pedestrian volume, as may be found in downtown areas, may significantly increase vehicular delays at unsignalized intersections. This increase stems from the additional con-

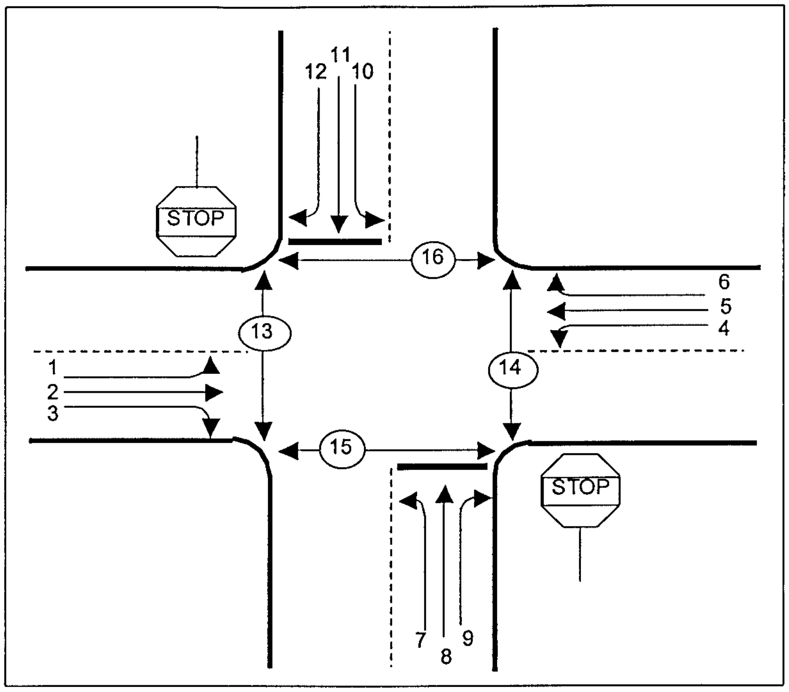


Figure 2-12. Movement numbers for right-turn volume adjustment.

flict between vehicular and pedestrian flows at the intersection. Because of this interaction, an intersection with heavy pedestrian volume is likely to derive more benefit from signalization than one where there are fewer pedestrians.

The *Highway Capacity Manual 2000* (*15*) procedure for two-way stop-controlled intersection analysis accounts for pedestrians by including them in the number of events (vehicle or pedestrian) that conflict with the non-priority movements. This approach can also be used to determine if pedestrian volume might mislead the warrant check. Specifically, it is suggested that a second warrant check be performed with the following adjusted volumes:

$$\text{Adjusted Major Road Volume} = V_1 + V_2 + V_3 + V_4 + V_5 + V_6 + V_{15} + V_{16}$$

$$\text{Adjusted Minor Road Volume} = \text{Larger of: } (V_7 + V_8 + V_9, V_{10} + V_{11} + V_{12}, V_{13}, V_{14})$$

where

V_i = movement volume shown in Figure 2-12.

Also, if the adjusted minor-road volume is based on a pedestrian volume, then the approach should be considered to have one approach lane.

If the second warrant check yields different conclusions than the first warrant check (i.e., the check based on unadjusted volumes), then pedestrian volumes may be enough to affect the accuracy of the warrant check. In this situation, it is recommended that the effect of pedestrians be fully examined during the engineering study.

Progressive Traffic Flow on the Major Road

Signal coordination promotes efficient traffic movement along the roadway by providing for the uninterrupted progression of through vehicle platoons. These platoons affect the operation of an unsignalized intersection. If platoons arrive simultaneously, there is a positive effect. Conversely, if platoons arrive in an alternating pattern, the effect is negative. The extent of the effect typically increases as the distance between intersections decreases.

Chapter 17 of the *Highway Capacity Manual 2000* (*15*) indicates that the effect of an upstream signal decreases as the distance between it and the subject intersection increases; the effect is negligible for distances in excess of 400 m (1,300 ft). Based on this guidance, platoon progression may affect the accuracy of the conclusions from the warrant check when the subject intersection is within 400 m (1,300 ft) of an upstream signalized intersection. If the analyst is uncertain of the effect of progression on the existing unsignalized intersection, then the effect of platoon progression should be fully evaluated during the engineering study.

Three-Leg Intersection

The performance of a two-way stop-controlled intersection can be greatly affected by the number of minor-road approach legs. A four-leg intersection has higher delay per vehicle than one with three legs because of the increased number of conflicting movements. Therefore, the benefit derived by signalizing a three-leg intersection may be substantially less than that derived by signalizing a four-leg intersection.

A simulation study conducted by Saka (*27*) indicated that *MUTCD 2000* Signal Warrant 1 (Condition A) has threshold volumes that are relatively low when applied to three-leg intersections. Saka recommended that a minimum major-road volume of 1,000 veh/h (total of both directions) and a minimum minor-road volume of 200 veh/h should be sustained for the 8 highest hours of the average day before the traffic signal alternative is considered for a three-leg intersection. If this volume level is not sustained, but Signal Warrant 1 is satisfied, then it is possible that the conclusion from the warrant check is inaccurate. In this situation, the operation of the three-leg intersection should be fully evaluated during the engineering study.

Added Through Lane on the Major Road

The outside through lane should be sufficiently long in advance of and beyond the intersection that it is attractive to through drivers. A through lane will not be fully used when it extends only a short distance before or after an intersection. Instead, only turning drivers may use it. The consequences of this problem are significant if the analyst misjudges the utilization of the second lane of a two-lane approach.

Guidelines provided in Chapter 10 of the *Highway Capacity Manual 2000* (*15*) can be used to determine how a lane may function at a given intersection. These guidelines are based on the length of the lane, as measured along the intersection's approach and departure legs. The guidance is illustrated in Figure 2-13.

Three regions are shown in Figure 2-13. The region labeled "Full lane" indicates combinations of approach and departure lane length that should allow the traffic lane to operate effectively as a through lane in the vicinity of the intersection. The remaining two regions indicate lane lengths that would yield partial or no use of the lane by through drivers.

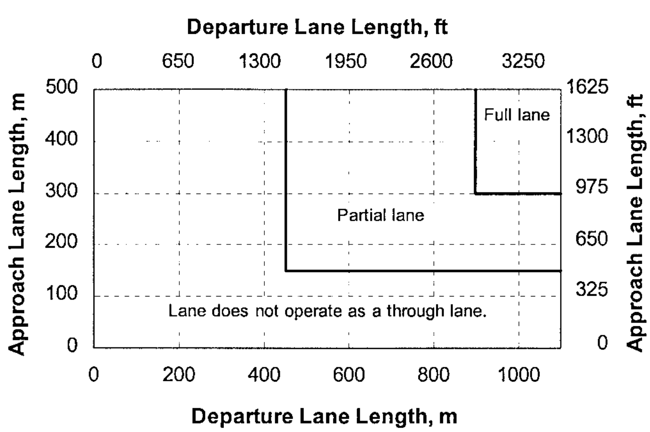


Figure 2-13. Effect of a lane's length on its utilization by through drivers.

The number of through lanes used in the warrant check should reflect fully utilized lanes to avoid inaccurate conclusions. If the number of lanes used in the warrant check does not reflect "fully used" lanes (as suggested by Figure 2-13), then it is possible that the conclusions reached from the warrant check will be inaccurate. Regardless, if the analyst is uncertain as to the effective number of through lanes at an intersection, then the effect of lane use should be fully evaluated during the engineering study.

Left-Turn Bay

The operation of the major- and minor-road traffic movements is affected by the approach lane assignments. Provision of an exclusive left-turn bay (of adequate length) separates the left-turn movement from the through movement and generally improves the operation of all movements, relative to a shared-lane arrangement. Therefore, an intersection that has shared lanes is likely to derive more benefit from signalization than an intersection that has one or more left-turn bays.

Guidance is provided in Section 4C.01 of the *MUTCD 2000* (*1*) as to how the effect of a left-turn bay can be addressed in the warrant check. For left-turn bays that serve about 50 percent of the approach traffic, it is suggested that the left-turn bay be included in the count of major-road (or minor-road) lanes. In contrast, for left-turn bays that serve a minor percentage of approach traffic, the *MUTCD 2000* (*1*) suggests that the left-turn bay should not be included in the lane count. In either case, the left-turn volume is included in the approach volume estimate.

Application of this guidance should improve the likelihood of accurate conclusions from the warrant check. However, if the analyst is uncertain whether to include the left-turn bay in the lane count, then the effect of the left-turn bay should be fully evaluated in an engineering study. Similarly, if an existing left-turn bay is believed to be too short, then it is possible that the conclusions reached from the warrant check will be inaccurate. In this situation, the benefit of the bay should be fully evaluated during an engineering study.

Right-Turn Bay on the Minor Road

The provision of an exclusive right-turn bay (or the availability of sufficient approach width to facilitate operation of a de facto exclusive right-turn bay) can significantly improve the operation of the minor-road right-turn movement, relative to a shared-lane arrangement. Therefore, an intersection that has shared lanes is likely to derive more benefit from signalization than an intersection with an exclusive right-turn bay on the minor-road approach. A method for determining whether the provision of a right-turn bay will affect the accuracy of the warrant check was described earlier in this chapter in the subsection, Right-Turn Volume on the Minor Road.

Wide Median on the Major Road

An unsignalized intersection with a major-road median of sufficient width to accommodate (or store) one or more crossing vehicles may operate as two separate intersections, each with a one-way major road. The frequency of such "two-stage" crossing maneuvers increases with increasing median width and with the willingness of drivers to cross in two stages.

The procedures in Chapter 17 of the *Highway Capacity Manual 2000* (*15*) indicate that two-stage operation can double the capacity of the minor-road traffic movements (even when the median is only 5 m in width). Therefore, in areas where drivers frequently cross in stages, an intersection with a wide median is less likely to benefit from signalization than one where there is no median.

Intersections with median widths of 5 m (16 ft) or more may be associated with inaccurate warrant check conclusions if two-stage crossing behavior occurs with any frequency. Therefore, when such conditions are believed to exist, the potential for and the effect of two-stage crossings should be fully evaluated during the engineering study.

Combination of Factors

The preceding discussion of problematic conditions is focused on describing individual factors that may influence the accuracy of the warrant check conclusions. The discussion associated with each condition was primarily based on the assumption that only that condition would be present at the subject intersection. Additional care should be taken when two or more problematic conditions are present at an intersection because interactions among these conditions could amplify their effect on traffic operations. More important, the existence of two or more problematic conditions heightens the need for a thorough evaluation during the engineering study.

CHAPTER 3

ENGINEERING STUDY

This chapter describes the engineering study stage of the engineering assessment process. During this stage, the various alternatives identified in the alternative identification and screening stage are evaluated in terms of their performance. Those alternatives that offer some level of improvement, relative to the existing intersection, are then advanced to the alternative selection stage. The alternative selection stage is the subject of the next chapter.

A procedure for evaluating the operational efficiency of intersection improvement alternatives is presented in this chapter. The presentation is divided into two sections. In the first section, the procedural steps involved in the operational assessment process are defined. The second section provides guidelines for designing the signalized intersection alternative and for using stochastic simulation models. The assessment of an alternative's other impacts (e.g., safety) is an important element of a comprehensive engineering study; however, procedures to guide these assessments are outside the scope of this guide.

PROCESS

Overview

The procedure for conducting an operational evaluation of an intersection (and its potential improvement alternatives) consists of three steps. These steps, described in this chapter; are as follows:

1. Determine study type,
2. Select analysis tool, and
3. Conduct evaluation.

In the first step, issues are addressed that define the nature of the study to be conducted. In the second step, the analyst identifies an analysis tool on the basis of consideration of traffic, geometry, and control conditions at the subject intersection and its alternatives. Finally, in the third step, the analyst develops the alternatives to a preliminary design level and then uses the selected analysis tool to quantify the operational performance of the subject intersection and each alternative.

The objective of the engineering study stage is to determine if a proposed improvement alternative is justified. An alternative is justified if an analysis indicates that (1) it provides a solution to the problem that precipitated the need for the study and (2) all traffic movements will operate safely and efficiently. The objective of determining if a proposed improvement alternative is justified or not is achieved through a process of designing and evaluating the viable alternatives identified in the alternative identification and screening stage. The steps involved in this process are described in the remainder of this section.

Step 1. Determine Study Type

The first step in the engineering study stage requires the consideration of two issues that characterize the type of study needed for the subject intersection. A procedure for resolving each issue is described in a separate task within this step. The two tasks are

a. Determine type of operation and
b. Determine type of evaluation.

In the first task, the relationship between the subject intersection and any upstream or downstream signalized intersection is evaluated. Then, in the second task, the complexity of each alternative is assessed, as is the corresponding level of analytical detail needed for its evaluation. Together, these determinations are used by the analyst to determine the best balance between the thoroughness of the study and the degree of certainty needed in its findings.

1-a. Determine Type of Operation

Overview. The first step in the engineering study is to characterize the effect of an upstream signalized intersection on the operation of the subject intersection. The existence of nearby signalized intersections adds a level of complexity to the analysis process because the platoons from these intersections can affect the operation of the subject intersection. Moreover, if a signal alternative is proposed for the subject intersection, then signal timing strategies that promote progression must also be included in the development of this alternative.

Isolated or Non-Isolated. The nature of the evaluation to be conducted is dependent on whether the subject intersection operates in isolation or within a signal system. The evaluation of an intersection that lies within a signal system (i.e., a non-isolated intersection) requires consideration of the effects of the upstream signalized intersections. In contrast, the evaluation of an isolated intersection does not have this requirement.

Figure 3-1 can be used to characterize the type of operation at the subject intersection. This figure was developed by Bonneson and Fontaine (*20*) and is based on the “coupling index” concept described by Orcutt (*28*) and by Hook and Albers (*29*). They used this concept to define road segment volumes and lengths that are likely to benefit from signal coordination. It is used in this document to define the degree of isolation at the subject intersection.

Use of Figure 3-1 requires identification of the road segment length and volume. The road segment length is the distance between the subject intersection and the upstream signalized intersection. The road segment volume is the peak-hour two-way volume flowing between these two intersections. All alternatives can be considered as “isolated” if the road segment length exceeds 1600 m (5,200 ft).

Three regions are identified in Figure 3-1. Each region characterizes traffic platoons in terms of their size and concentration. The region labeled as “Sizeable platoons” reflects very concentrated and lengthy platoons on relatively short road segments. If the subject intersection is associated with volume and length values that fall in this region, then it should be considered a non-isolated intersection. All alternatives associated with this intersection should reflect consideration of platoon effects. If the traffic signal alternative is being considered, it should be coordinated with the upstream signal.

The region labeled as “Moderate platoons” in Figure 3-1 reflects the likelihood of sizeable platoons over a wide range of segment lengths. If the subject intersection is associated with volume and length values that fall in this region, then it should be considered a non-isolated intersection. The effect of traffic platoons on unsignalized intersection operation is more subtle in this region than that associated with the “Sizeable platoon” region. Hence, alternatives associated with unsignalized control should not require consideration of platoon effects. However, if the traffic signal alternative is being considered, it should be coordinated with the upstream signal.

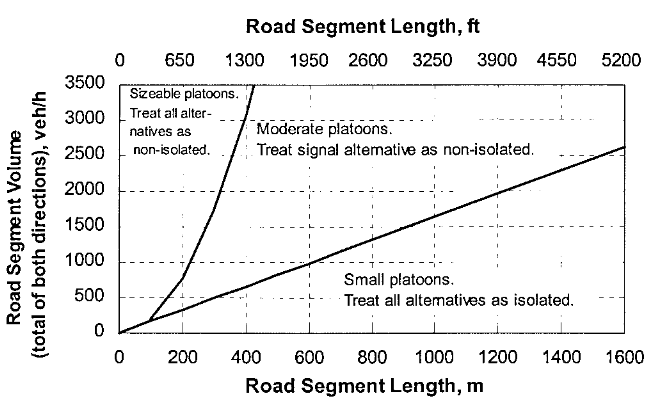


Figure 3-1. Volume and segment length combinations that define isolated and non-isolated operation.

Finally, the region labeled as “Small platoons” in Figure 3-1 reflects the likelihood of short and dispersed platoons on low-volume roads. If the subject intersection is associated with volume and length values that fall in this region, then it should be considered an isolated intersection. Alternatives associated with the subject intersection do not need to reflect consideration of platoon effects. If the traffic signal alternative is being considered, it does not need to be coordinated with the upstream signal for the purpose of the engineering study.

In application, all approaches to the subject intersection that have upstream signals would be checked using Figure 3-1. If any one approach is found to be “non-isolated,” then the intersection should be evaluated as non-isolated. In addition, if the agency wants to include coordination with the signal alternative (independent of the guidance from Figure 3-1), then the subject intersection should be evaluated as non-isolated.

1-b. Determine Type of Evaluation

Overview. During this task, the engineer determines the type of evaluation needed for the engineering study. Two types of evaluation are possible: (1) the formal engineering study process as described in the remainder of this chapter and (2) the deployment and field evaluation of a proposed alternative. This latter type of evaluation is referred to as “informal” because it does not follow the formal engineering study process. The conditions where an informal evaluation is appropriate are described in the next section.

Formal Versus Informal Evaluation. The informal (or field) evaluation of an alternative represents a practical method of assessing an alternative’s performance. This type of evaluation is most appropriate for intersections that are effectively isolated from the effects of upstream signals and that have typical traffic and geometric features. Several conditions should be met before an informal evaluation is considered. These conditions are

1. The intersection is isolated,
2. Only one alternative is viable,
3. Geometric and traffic conditions are typical and consistent with the assumptions underlying the warrant or guideline used to verify the alternative’s viability (see discussion of problematic conditions in Chapter 2 if signal control is the one alternative), and
4. The engineer’s experience and judgment indicate that the alternative would resolve the operational problem that precipitated the study without creating additional problems.

If the aforementioned conditions are satisfied, then the engineer may choose to forego the remaining steps in the formal engineering study process. In this case, the engineering study would consist of a field evaluation of the installed alternative. The field evaluation would verify that the alternative solved the operational problem that precipitated the study and did not degrade the overall operation of the intersection. If one or more of the four conditions for an informal evaluation are not satisfied, then the formal engineering study should be conducted.

Step 2. Select Analysis Tool

During this step, the analyst will identify the capacity analysis procedure or simulation model (hereafter referred to as "analysis tool") that is most appropriate for the engineering study. A procedure for making this identification is described in this section. The procedure consists of the following two tasks:

a. Identify desired capabilities and
b. Evaluate and select analysis tool

During the first task, the desired functions and output capabilities of the analysis tool are identified. Then, these functions and capabilities are compared with those of available analysis tools to determine which tool will be the most effective.

2-a. Identify Desired Capabilities

Overview. The analysis tool used for the engineering study must be able to model the traffic control modes and the traffic flow interactions found at the subject intersection and its alternatives. The tool must also be able to model a system of signalized and unsignalized intersections when the subject intersection is non-isolated. Finally, the analysis tool should also be able to predict the desired measures of effectiveness. At the conclusion of this task, the analyst will have developed a list of desired capabilities that can be used in Task 2-b to select the most appropriate analysis tool.

Any analysis tool used by the agency should be calibrated to replicate existing conditions and be verified for accuracy. However, formal calibration of an analysis tool for any one engineering study may not be cost-effective. Instead, calibration should be an annual (or bi-annual) undertaking based on data obtained from a comprehensive study of traffic conditions in the agency's jurisdiction.

Traffic Control Modes. The traffic control modes that are considered by an analysis tool should include that of the existing intersection and any proposed alternatives. Table 3-1 lists the control modes that are often required in engineering studies. The analyst should review this list and indicate by checkmark (✓) those modes that will be needed for the study of interest.

Analysis Factors. The analysis tool selected for the engineering study should be able to model the various analysis factors relevant to the subject intersection. These factors include (1) the ability to accept important traffic characteristics as inputs and reflect their influence on traffic operations, (2) the ability to model key traffic interactions, and (3) the ability to accept important descriptors of intersection geometry and reflect their influence on traffic operations. The need to have any one of these factors in a particular model is dependent on conditions present at the subject intersection as well as those conditions present in the alternatives under consideration. Table 3-1 lists analysis factors that are often required for engineering studies. The analyst should review this list and indicate by checkmark (✓) those analysis factors needed for the study of interest.

Measures of Effectiveness and Performance Indicators. The analysis tool selected for the engineering study should be able to predict the various measures of effectiveness and performance indicators relevant to the subject intersection. The need to have any one of these attributes in a particular model is dependent on the types of operational problems being experienced and the performance assessment procedures of the agency conducting the study. Table 3-1 lists the measures and indicators that are often required for engineering studies. The analyst should review this list and indicate by checkmark (✓) those measures and indicators needed for the study of interest.

An important output measure of effectiveness is average-delay-per-vehicle. This measure is widely recognized by engineers and the motoring public as reflective of intersection performance. It is important that the analysis tool provide this statistic. For intersections with heavy transit or pedestrian flow, average-delay-per-person is another important statistic.

2-b. Evaluate and Select Analysis Tool

Overview. This section describes a framework for analysis tool selection. Initially, a case is made that only one analysis tool should be used for the evaluation. Following this discussion, a step-by-step tool selection process is outlined. It is recognized that some agencies may have guidelines regarding the analysis tool to be used in the engineering study. However, the analyst should still complete this step and confirm that the tool used in the study is appropriate for the analysis of the subject intersection and its alternatives.

One Analysis Tool. This section describes two requirements that the analysis tool should satisfy to be used in the engineering study. First, the analysis tool should be able to *explicitly* model the operational features of the intersection and, if needed, the associated street system. This requirement recognizes the need for accuracy in the evaluation. It is intended to discourage the creative extension of tools to situations that they are not specifically developed to model. Thus,

TABLE 3-1 Analysis tool capabilities

Feature	Category	Capability	Need?
Traffic Control Mode	Unsignalized	Two-way stop control	
		Two-way yield control	
		Multi-way stop control	
		Roundabout	
	Signalized	Pretimed signal control	
		Actuated signal control	
		Signal coordination	
Analysis Factors	Traffic Characteristics	Vehicle occupancy	
		Pedestrian volume	
		Percentage of heavy vehicles	
		Parking maneuvers	
		Bus stop frequency	
		Approach speed	
		Duration of study period / oversaturated conditions	
	Traffic Modeling	Platoon progression modeling	
		Two-stage crossing at two-way stop-controlled intersection	
		Right-turn-on-red at signalized intersection	
		Left-turn bay overflow	
		Delay to unstopped through vehicles due to turning maneuvers	
		Spillback from a downstream signal	
	Geometry	Grade	
		Exclusive left-turn lane	
		Exclusive right-turn lane	
		Shared lanes	
		Right-turn radius	
		Downstream through lane length (i.e., lane drop)	
		Multi-leg intersections (i.e., five or more approaches)	
Output Measures and Indicators	Measure of Effectiveness	Average delay per person	
		V/C ratio (degree of saturation)	
		Total delay	
		Average delay per vehicle	✓
		Fuel consumption/emissions	
		Average queue length	
		Max. backward extent of queue	
		Travel speed (or travel time)	
		Stop rate	
	Performance Indicator	Graphical animation of system	
		Level of service	
		Time space diagram	
		Total operating cost	

if a coordinated semi-actuated control alternative is being considered, the analysis tool should explicitly model platoon progression, coordination, and actuated control.

Second, the delay statistics used to compare various alternatives should be consistently defined and computed. This requirement promotes the accurate evaluation of the relative impact of each alternative on intersection operations. Thus, if traffic signal control is being considered as an alternative, the delay computed for both the existing intersection and the signalized intersection should be based on a common definition (e.g., control delay) and similarly derived equations.

Based on this discussion, it follows that one analysis tool that can model all of the alternatives should be used for the engineering study. This approach would ensure equitable comparisons among alternatives by avoiding biases that might be present when two or more tools are used. Such bias could result from differences in modeling approach or delay definition. The use of one tool may also offer some analysis efficiency due to common input and output data formats.

Selection Process. The analysis tool selection process is based on the principle that the analysis tool selected should provide an equitable balance between analysis complexity, accuracy, and cost. It is also based on the assumption that one analysis tool will be used for all evaluations. The process consists of the following four activities:

1. Identify applicable traffic control modes,
2. Identify applicable traffic and geometric factors,
3. Identify applicable output measures and performance indicators, and
4. Select analysis tool.

These activities are described in the following paragraphs. Tables 3-2, 3-3, and 3-4 are consulted during this task. Collectively, these tables identify the capabilities of the various analysis tools used in the engineering study. One table column should be allocated for each candidate analysis tool. The tool-specific information entered into these columns will need to be provided by the analyst. The first column in each of the three tables has been completed (for a hypothetical analysis tool) to illustrate the manner in which relevant information can be recorded.

Some effort will be required initially to complete these tables. However, once they are completed, the development effort will be offset by a significant time savings during subsequent engineering studies. One benefit of this formal tabulation of tool capabilities is that it ensures that all relevant modes, factors, and outputs have been considered (and accommodated to the extent possible) before any effort is expended in the evaluation process. These tables should be periodically updated to maintain their effectiveness.

Activity 1: Identify Applicable Traffic Control Modes. The control modes identified in Task 2-a are cross-checked with the tool capabilities identified in Table 3-2. Those analysis tools that provide the desired capability should be identified and advanced to Activity 2.

To illustrate the use of Table 3-2, consider a situation where the existing, isolated intersection has two-way stop-control and where actuated signal control is considered to be a viable alternative. Based on the example information provided in this table, it appears that ABC software is able to model both control conditions and can be advanced to Activity 2. In contrast, if the intersection were non-isolated or if two-way yield control were an alternative, then this software would not be appropriate because it cannot model either of these control modes.

Activity 2: Identify Applicable Traffic and Geometric Factors. The second activity in the selection process is to cross-check the analysis factors identified in Task 2-a with the tool capabilities identified in Table 3-3. Those analysis tools that provide the desired capability should be identified and advanced to Activity 3. (Only those tools that have the desired control mode *and* analysis factor capabilities are advanced to the next activity.)

To illustrate the use of Table 3-3, consider a situation where the existing, isolated intersection has two-way stop-control and where the addition of an exclusive left-turn lane on the minor road is a viable alternative. Based on the example information provided in this table, it appears that ABC software can model the effect of adding a left-turn lane and may be appropriate for the analysis and should be advanced to Activity 3. However, this software is not sensitive to the operational problems that would result from a left-turn bay

TABLE 3-2 Traffic control mode check sheet

Traffic Control Mode		Analysis Tool				
		Isolated Only			Non-Isolated	
		ABC software				
Unsignalized	Two-way stop control	✔				
	Two-way yield control	--				
	Multi-way stop control	✔				
	Roundabout	✔				
Signalized	Pretimed signal control	✔*D*				
	Actuated signal control	✔				
	Signal coordination	--				

Notes:
✔ - Tool explicitly models this control mode and its effect.
-- - Tool does not explicitly model this control mode or its effect.
D - Tool can determine the optimal signal phase duration thereby eliminating the need for signal timing plan input.

TABLE 3-3 Analysis factor check sheet

Category	Factor	Analysis Tool				
		Isolated Only			Non-Isolated	
		ABC				
Traffic Characteristics	Vehicle occupancy	*S, T, M*				
	Pedestrian volume	*S, T*				
	Percentage of heavy vehicles	*S, T, M*				
	Parking maneuvers	*S*				
	Bus stop frequency	*S*				
	Approach speed	--				
	Duration of study period / oversaturation	*S, T*				
Traffic Modeling	Platoon progression modeling	*S, T*				
	Two-stage crossing at 2-way stop-control int.	*T*				
	Right-turn-on-red at signalized intersection	--				
	Left-turn bay overflow	--				
	Delay to unstopped throughs due to turning	*T*				
	Spillback from a downstream signal	--				
Geometry	Grade	*S, T*				
	Exclusive left-turn lane	*S, T*				
	Exclusive right-turn lane	*S, T*				
	Shared lanes	*S, T, M*				
	Right-turn radius	--				
	Downstream through lane length (lane drop)	--				
	Multi-leg intersections (i.e., 5 or more app.)	*S, T, M*				

Notes:
T - Tool considers this factor and its effect for two-way stop control.
M - Tool considers this factor and its effect for multi-way stop control.
S - Tool considers this factor and its effect for signal control.
-- - Tool does not explicitly consider this factor or its effect.

overflow. Thus, if the left-turn bay were restricted to a relatively short length, this software would not be appropriate because it cannot model turn bay overflow.

Activity 3: Identify Applicable Output Measures and Performance Indicators. The third activity in the selection process is to cross-check the output measures and indicators identified in Task 2-a with the tool capabilities identified in Table 3-4. Those analysis tools that provide the desired capability should be identified and advanced to Activity 4. (Only those tools that have the desired control mode, analysis factor, and output capabilities are advanced to the next activity.)

Activity 4: Select Analysis Tool. The fourth activity in the selection process is to review the list of candidate analysis tools and select the one tool that has the lowest resource costs. These costs include data entry, software maintenance, training, and model calibration. A high degree of precision in this estimate is not essential. To aid in this assessment, the analyst may wish to seek the opinion of other engineers who have experience using one or more of the analysis tools.

Step 3. Conduct Evaluation

The third step in the engineering study stage focuses on quantifying the operational performance of the existing intersection and any proposed improvement alternatives. The tasks involved in this evaluation process are discussed in this section; they are

a. Gather information,
b. Evaluate operational performance, and
c. Determine alternative effectiveness.

In the first task, relevant traffic, signalization, and geometric data are identified and collected. Then, these data are used with the analysis tool identified in Step 2 to evaluate the subject intersection and any proposed alternatives.

3-a. Gather Information

Overview. There are two objectives of this task. The first objective is to define the evaluation time periods. The second

TABLE 3-4 Measures of effectiveness and performance indicators check sheet

Category	Output Information	Analysis Tool				
		Isolated Only			Non-Isolated	
		ABC				
Measure of Effectiveness	Average delay per person	--				
	V/C ratio (degree of saturation)	*S, T*				
	Total delay	--				
	Average delay per vehicle	*S, T, M*				
	Fuel consumption/emissions	--				
	Average queue length	--				
	Max. backward extent of queue	*S, T*				
	Travel speed	--				
	Travel time	--				
	Stop rate	--				
Performance Indicator	Graphical animation of system	--				
	Level of service	*S, T, M*				
	Time space diagram	--				
	Total operating cost	--				

Notes:
T - Tool explicitly computes and reports this measure for two-way stop control.
M - Tool explicitly computes and reports this measure for multi-way stop control.
S - Tool explicitly computes and reports this measure for signal control.
-- Tool does not explicitly compute and report this output measure.

objective is to identify the data needed for Task 3-b, Evaluate Operational Performance. At the conclusion of this task, all of the data needed for the evaluation should be identified and collected.

Define Evaluation Periods. At the start of this task, the periods for which traffic operations will be evaluated should be defined. It is generally sufficient to evaluate operations for three representative 1-hr periods. These hours should include the morning and afternoon peak traffic demand hours as well as one representative off-peak demand hour. The off-peak hour should represent an average of traffic conditions during the hours from 6:00 a.m. to 10:00 p.m., excluding the two peak hours. Additional hours can be evaluated if traffic patterns are highly varied or if a more comprehensive assessment of operations is required.

Identify Data. Much of the data needed for Task 3-b will have been collected or derived during the alternative identification and screening stage (see Chapter 2). However, additional data may be needed to calibrate the analysis tool selected in Task 2-b. Also, the data previously collected may be incomplete in terms of the evaluation periods just defined. Data that may need to be collected include

- Turning movement counts,
- Pedestrian and heavy vehicle counts,
- Approach grade,
- Approach speed, and
- Vehicle occupancy.

If the subject intersection is non-isolated, then additional data will be needed to characterize the operation of the signal system. Such data may include

- Turning movement counts at adjacent signalized intersections,
- Phase sequence and duration at adjacent signalized intersections,
- Signal offset relationship between signalized intersections, and
- Geometry of adjacent signalized intersections.

Collect Data. The procedures for collecting the necessary data will vary depending on whether the subject intersection exists or is proposed for construction. For existing intersections, appropriate data collection procedures are described in the *Manual of Transportation Engineering Studies* (*6*). For proposed intersections, techniques described in Appendix C can be used to estimate turn movement volumes from forecast average daily traffic demands. Regardless of the source, the data should represent traffic conditions occurring on an "average day" (i.e., a day representing traffic volumes normally and repeatedly found at a location).

3-b. Evaluate Operational Performance

Overview. The subject intersection and all viable alternatives should be formally evaluated during this task. This evaluation should determine the average delay to each intersection traffic movement, as well as the overall average intersection delay. These delay measures will be used in Task 3-c to determine whether an alternative can effectively improve the operation of the subject intersection.

Design Alternatives. Before the evaluation, the physical elements of each alternative should be sized and the functional elements identified (at a preliminary design level of detail) to facilitate an accurate evaluation of the alternative's operational effectiveness. The time expended in this design effort will increase with the alternative's complexity; however, the consequences of not devoting the time needed for this design can be far more costly. For example, an overly simplified signalized intersection design may yield inaccurate performance estimates and could result in the analyst recommending what will ultimately turn out to be an "unnecessary" signal.

With one exception, preliminary design guidelines for intersection improvement alternatives are not provided in this document. The analyst is directed to other documents, such as AASHTO's *A Policy on Geometric Design of Highways and Streets* (*Green Book*) (*5*) or the agency's roadway design manual, for this guidance. The one exception is the traffic signal alternative. Guidelines for the design of this alternative are provided in the Guidelines section of this chapter.

Conduct Evaluation. At this point, the analyst should use the analysis tool identified in Step 2 to evaluate the operation of the subject intersection and the various alternatives identified in the alternative identification and screening stage.

Analysis tools that are characterized as "stochastic" will require additional effort on the part of the analyst to ensure that the output delay statistics are accurately interpreted. This requirement results from the stochastic analysis tool's explicit modeling of traffic events and driver decisions as random processes. This modeling approach has the advantage of producing a great deal of realism in the simulation of the traffic system; however, it has the disadvantage of requiring extra effort in setting up and running the simulation model. Guidelines for using stochastic analysis tools in the engineering study are provided in the Guidelines section of this chapter.

3-c. Determine Alternative Effectiveness

Overview. The objective of this task is to determine if any of the alternatives (including the "do nothing" alternative) evaluated in Task 3-b can serve traffic effectively. Criteria are described in this task for making this determination. Initially, two descriptors of traffic service quality are defined. Then, these descriptors are used to describe the conditions that an alternative must satisfy to be termed "effective." At the conclusion of this task, all effective alternatives are identified and advanced to the alternative selection stage (see Chapter 4).

Definitions. Two descriptors are used to define threshold conditions that, if exceeded, are associated with unacceptable operations. These two descriptors are "acceptable level of service" and "acceptable operation." Their definitions are

- *Acceptable level of service*—an overall intersection operation that can be described as level-of-service (LOS) D or better [as defined in the *Highway Capacity Manual 2000* (*15*)].
- *Acceptable operation*—an intersection traffic movement that can be described as operating at (1) LOS D or better or (2) a total delay less than 4.0 vehicle-hours (5.0 vehicle-hours for a multilane movement).

The "total delay" threshold (i.e., 4.0 or 5.0 vehicle-hours) is based on recommendations made by Henry and Calhoun (*26*) and by Sampson (*30*) as a result of their signal-warrant-related research. The "level-of-service" threshold (i.e., "D") is based on the recommendation made in Table II-6 of the *Green Book* (*5*). The analyst may substitute agency-preferred thresholds.

The effect of considering both total delay and level-of-service (defined in terms of average control delay) is illustrated in Figure 3-2. The convex trend line shown in this figure represents volume and delay combinations that equal 4.0 (or 5.0) vehicle-hours of total delay. The horizontal trend line represents 35 seconds per vehicle (s/veh) of average delay. This level of delay represents an upper limit for LOS D at unsignalized intersections [based on definitions in the *HCM 2000* (*15*)]. The thick line represents the combination of total delay and average control delay that form an upper limit on "acceptable operation." Because the curved trend line is above that associated with LOS D for volumes less

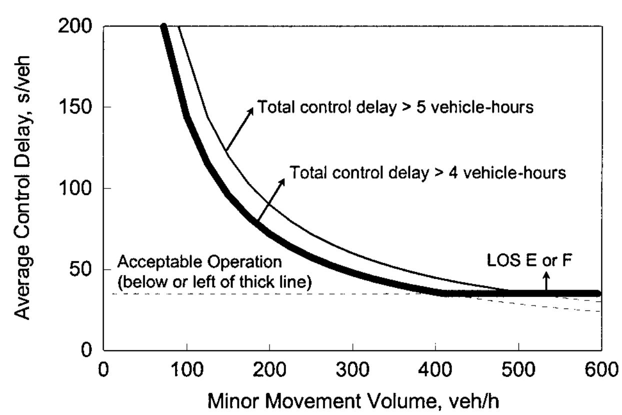


Figure 3-2. Acceptable operating conditions at unsignalized intersections.

than about 420 vehicle-hours, a low-volume movement may have delays that yield LOS E or F but may still be considered as having "acceptable operation" by this approach.

Conditions for Identifying Effective Alternatives. Conditions are described in this section that can be used to identify alternatives that are operationally effective and, therefore, suitable for consideration in the alternative selection stage. A separate series is provided for the isolated and non-isolated intersection types. Each condition must be satisfied before an alternative can be considered effective. The conditions for each intersection type follow.

For Isolated Intersections. An effective alternative should

1. Improve overall intersection operations by reducing average intersection delay (based on an assessment of conditions existing throughout the typical day); and
2. Result in an "acceptable level of service" for the intersection (based on an assessment of the peak traffic demand hour); and
3. Result in the "acceptable operation" of all minor movements (based on an assessment of the peak traffic demand hour).

For Non-Isolated Intersections. An effective alternative should

A. Improve overall system operations by reducing the average system delay (based on an assessment of conditions existing throughout the typical day); and
B. Result in an "acceptable level of service" for each signalized intersection in the system (based on an assessment of the peak traffic demand hour); and
C. Result in the "acceptable operation" of all minor movements at each signalized intersection (based on an assessment of the peak traffic demand hour); and
D. Satisfy Conditions 1, 2, and 3 (as described immediately above) for the subject intersection.

Four terms or phrases used in the preceding conditions are defined as follows:

- The "average intersection delay" identified in Condition 1 should be computed as a volume-weighted average that reflects all approach traffic movements at the intersection.
- The "average system delay" identified in Condition A should be computed as a volume-weighted average that reflects all approach movements at each intersection in the system.
- The phrase "conditions existing throughout the day" used in Conditions 1 and A means conditions occurring during the morning and afternoon peak hours as well as one off-peak hour; additional hours can be included in the assessment to provide better representation.
- As a minimum, the "system" identified in Conditions A and B should include the nearest upstream and downstream signalized intersections. Additional signalized intersections along the arterial street may also be included, if the analyst determines that they would be affected by improvements to the subject intersection.

In application, these conditions would be assessed for each alternative. If all conditions are satisfied for a given alternative, then the alternative is "effective" in terms of being able to improve operations at the subject intersection. All "effective" alternatives should be advanced to the alternative selection stage, as described in Chapter 4.

GUIDELINES

This section provides guidance related to select components of the engineering study stage. The guidance is presented in the following two sections:

1. Guidelines for Designing the Signalized Intersection Alternative and
2. Guidelines for Use of Stochastic Simulation Models.

The first section describes guidelines that can be used to make preliminary design decisions for the traffic signal alternative. The second section describes guidelines that can be used when the analysis tool simulates traffic flow as a random process such that the predicted performance measures (e.g., delay) vary with each simulation run.

Guidelines for Designing the Signalized Intersection Alternative

Overview

This section describes guidelines for the preliminary design of a signalized intersection. The guidelines were obtained primarily from reference documents that are generally recognized as authoritative guides on engineering design. Other sources for the guidelines include reports documenting significant research efforts whose recommendations are intended for nationwide application. Where guidelines are not available, they have been derived using traditional engineering analysis techniques. The guidelines address the following design topics:

- Intersection geometry,
- Controller operation,
- Controller phase sequence,
- Basic controller settings,
- Controller settings for isolated operation, and
- Controller settings for coordinated operation.

These guidelines should be used to design the signalized intersection alternative if, during the alternative identification and screening stage, it is determined that the traffic signal alternative is viable. These guidelines are intended to assist the engineer to develop a reasonable representation of the traffic signal alternative. In this regard, the traffic signal alternative should be properly designed before being evaluated because the level of service it provides is often very sensitive to the selected phase sequence, signal timing, and geometry. If not properly designed, the traffic signal alternative's performance could be grossly overestimated or underestimated.

The guidelines provided in this section do not address all aspects of the preliminary design of a signalized intersection. Although they are believed to be appropriate for the more typical signalized intersections, some judgment may be needed in the design of intersections that have unique operating or geometric conditions. **These guidelines are intended to describe reasonable design choices solely for the purpose of fairly evaluating the traffic signal alternative; they are not intended to be used as a substitute for applicable state or local design standards.**

Several software products are available to automate elements of the signal timing design process. Some products can also be used to make geometric design decisions in a fairly efficient manner. The analyst may substitute these products for the guidelines offered in this section. The only stipulation is that each of the "bulleted" elements in the preceding list should be designed using sound, defensible engineering practices.

Intersection Geometry

This section describes guidelines for estimating the number of lanes needed for each intersection traffic movement. These guidelines were derived from the information provided in Appendix A of Chapter 16 in the *Highway Capacity Manual 2000* (*15*). These guidelines represent a good starting point for the intersection design. Once established, this idealized geometry can be modified to accommodate the lane configuration of the existing road system and the availability of right-of-way.

The guidelines are intended for separate application to each intersection approach and movement and should be used to estimate the number of lanes needed during both the morning and afternoon peak demand hours. Of these two estimates, the larger should be used to design the approach. If only one peak hour is evaluated, then the number of lanes determined for the through movement in the peak demand direction should also be used for the opposing through direction to ensure adequate service during both peak hours.

A minimum of two lanes should be considered for the minor-road approaches to an arterial street or highway. These two lanes may serve any combination of through or turn movements. This practice will minimize the effect of a new signal on the major-road operation (except when pedestrian crossing times dictate a lengthy minor-road phase duration). Provision of only one minor-road lane tends to result in an inequitable distribution of cycle time between major and minor movements. Such inequity is especially disruptive when the major road is coordinated.

Exclusive Left-Turn Lanes. The *Highway Capacity Manual 2000* (*15*) suggests that the following criteria can be used to estimate the number of lanes needed for the left-turn movement:

- Provide one or more exclusive lanes, if a left-turn phase is provided;
- Provide one exclusive lane, if 100 veh/h < V_{lt} < 300 veh/h; and
- Provide two exclusive lanes, if V_{lt} > 300 veh/h.

Where the variable V_{lt} represents the left-turn movement volume (in veh/h).

These volume thresholds are approximate because of the complex nature of shared-lane operations and are most appropriate when the opposing and adjacent through movements have flow rates of 450 veh/h or more. Exclusive turn lanes may not be needed for lower flow rates. The need for an exclusive left-turn phase can be determined using the guidance provided in the section, Left-Turn Phasing.

Through Lanes (or Shared Lanes). *Highway Capacity Manual 2000* (*15*) suggests that enough through lanes should be provided on an approach to ensure that the through volume (as well as any right- or left-turn volume sharing the through lanes) does not exceed 450 veh/h/ln.

Exclusive Right-Turn Lanes. Finally, the *Highway Capacity Manual 2000* (*15*) suggests that an exclusive right-turn lane should be considered when the right-turn volume exceeds 300 veh/h and the adjacent through lane volume exceeds 300 veh/h/ln.

The guidance provided in the previous two paragraphs is generalized in Figure 3-3. This figure indicates the number of

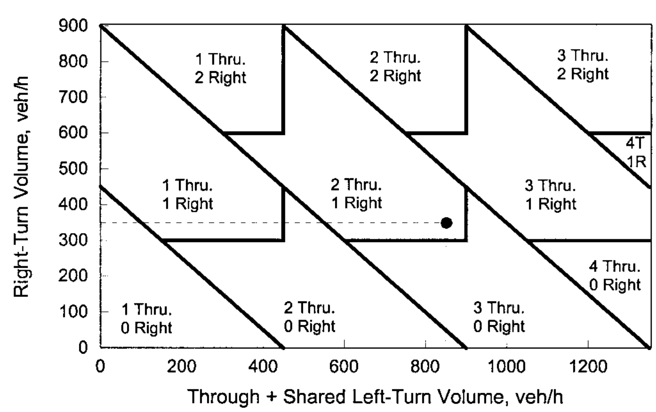


Figure 3-3. Relationship between approach volume and approach lane assignments.

through and right-turn lanes needed for various volume combinations. The *Highway Capacity Manual 2000* guidance has been extended for the development of this figure (e.g., it is assumed that right-turn volumes in excess of 600 veh/h may require two exclusive turn lanes).

The *x*-axis of Figure 3-3 represents the through volume on the subject approach as well as any left-turn volume that shares the through lanes (i.e., when no left-turn lane is provided). When consulting this figure, the analyst should find the point where the right-turn and the through (plus shared left-turn) volumes intersect. The location of this point determines the number of through lanes needed and, if appropriate, the number of exclusive right-turn lanes.

Example. To illustrate the use of Figure 3-3, consider an approach with the following flow rates during the peak demand hour:

- Left-turn volume - 50 veh/h,
- Right-turn volume - 350 veh/h, and
- Through volume - 800 veh/h.

The *Highway Capacity Manual 2000* (*15*) guidelines noted previously indicate that a left-turn volume of 50 veh/h is insufficient to justify a left-turn lane. Figure 3-3 is then checked using a "through + shared left-turn volume" of 850 veh/h (= 800 + 50) and a right-turn volume of 350 veh/h (see dashed lines). The figure indicates that two through lanes are needed along with one exclusive right-turn lane.

Reconciliation with Existing Roadways. After the intersection geometry is defined, it must be reconciled with that of the existing intersecting roadways. In general, the number of through lanes at an intersection should be consistent with the number of continuous through lanes on the roadway. An additional through lane at an intersection is generally not efficiently used unless it extends 900 m (3,000 ft) or more downstream; provision of such a length is often not practical. Similarly, it may not be practical to provide a dual left-turn lane when the receiving roadway has only one through lane.

Tradeoffs may be possible as the guidelines in this section are conservative. When the guidelines indicate that the intersection needs 10 to 20 lanes in total, one or two lanes can probably be excluded (if taken from a multilane movement) without causing major operational problems. Also, subtraction of one lane from a given traffic movement may be offset by adding a lane to a conflicting movement. However, significant changes in lane assignment or reductions in the total number of lanes may not be possible without causing a significant increase in delay. Ultimately, it is the responsibility of the engineer to define an intersection geometry that provides a reasonable balance between intersection operations and the geometry that can reasonably be accommodated by the existing roadways.

Controller Operation

This section provides guidelines for selecting the most appropriate control mode. These modes include pretimed, full-actuated, and semi-actuated control. The most appropriate choice of control mode for a given intersection is primarily based on the location of the subject intersection relative to nearby intersections and the type of signal operation used as a result of this location.

Determining the type of control that is best suited to a particular intersection is a fundamental and critically important design decision. Table 3-5 describes the range of application of the principal types of control. The guidance provided in this table was adapted from Gordon et al. (*31, p. 7–9*). The guidance regarding "consistent" versus "fluctuating" traffic demands that is provided in the table footnotes was obtained from Kell and Fullerton (*32, p. 31*).

TABLE 3-5 Relationship between intersection operation and control mode

Control Mode	Intersection Operation [1]		
	Isolated Intersection	Coordinated within Arterial Street System [2,3]	Coordinated within Network (or Grid) System
Pretimed	Usually not appropriate.	<u>Applicable if crossroad carries heavy and consistent traffic demands.</u>	<u>Prevalent type used</u>.
Semi-actuated	Usually not appropriate.	<u>Applicable if crossroad carries light or fluctuating traffic demands.</u>	Only applicable for isolated operation during light traffic periods.
Full-actuated	<u>Prevalent type used</u>.	Only applicable for isolated operation during light traffic periods.	Only applicable for isolated operation during light traffic periods.

Notes:
1 - <u>Underlined</u> text indicates combinations of control mode and intersection operation that are typically used.
2 - "Heavy" is defined as a crossroad demand that exceeds 20 percent of the arterial street demand. "Light" is defined as a crossroad demand that is less than 20 percent of the arterial street demand.
3 - "Consistent" demands can be assumed when only Warrant 1, Condition A is satisfied. "Fluctuating" demands can be assumed when only Warrant 1, Condition B is satisfied.

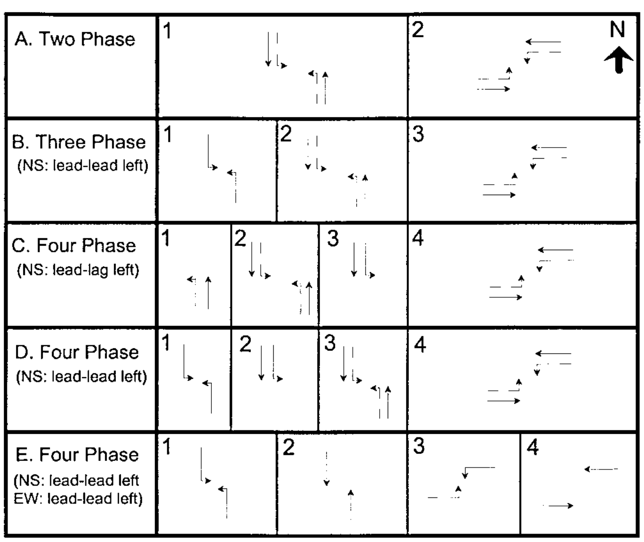
When local practice does not indicate the preferred control mode, the information in Table 3-5 can be used to identify a suitable mode. Application of this table requires that the identification of "intersection operation" and, in the case of arterial street systems, the character of the crossroad traffic demand. For example, the table indicates that an isolated intersection probably should have full-actuated control. On the other hand, an intersection that is located along an arterial street and that has "light" crossroad traffic demands probably should have semi-actuated control. Finally, an intersection located in a downtown street grid system probably should have pretimed control.

Controller Phase Sequence

This section describes several phase sequences often used at signalized intersections. By definition, a minimum of two phases is needed at an intersection of two roads. The provision of a third or a fourth phase sequence typically depends on whether the left-turn and opposing through movements need to be separated in time. If this separation is needed, it is typically accomplished by providing a left-turn phase. Guidelines are provided in this section that describe conditions where a left-turn phase may be needed.

Phase Sequence Variations. At a preliminary design level, the analyst should determine the manner in which the through and the left-turn movements will be served by the controller phase sequence. At the simplest level, two phases can be used to control the intersection. This two-phase sequence, shown as Sequence A in Figure 3-4, alternates the right-of-way between the two intersecting roads—one phase serving each road. If a road has two-way traffic, then its phase serves opposing through movements in an exclusive or "protected" manner. In contrast, the left-turning drivers are not protected and must yield to the oncoming through vehicles. This type of yielding left-turn service is referred to as "permitted" left-turn operation.

If gaps in the oncoming stream are inadequate to serve the left-turn demand or if left-turn-related accidents are unusually frequent, then a protected left-turn phase is often added to the phase sequence. This left-turn phase can occur before (i.e., lead), after (i.e., lag), or at the same time as the adjacent through-movement phase. Figure 3-4 illustrates several three- and four-phase sequences that include both through and left-turn phases.



Note: solid lines denote protected movements; dashed lines denote permitted movements.

Figure 3-4. Selected signal phase sequences.

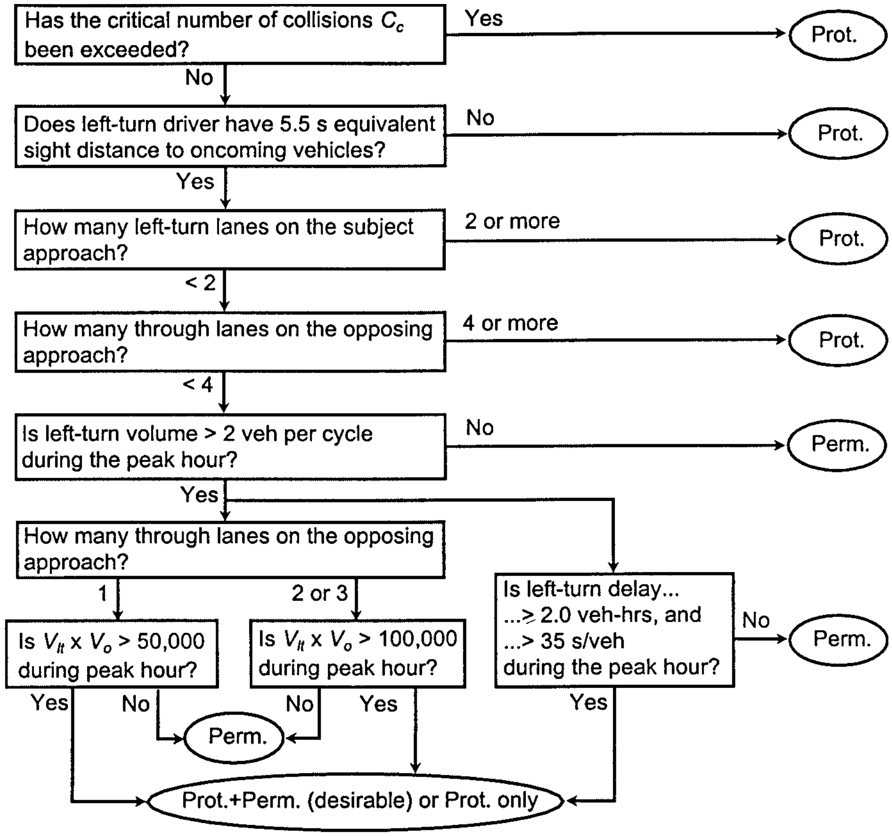
Left-Turn Phasing. As a general rule, the number of phases should be kept to a minimum because each additional phase in the signal cycle reduces the time available to the other phases. Thus, two-phase operation with permitted left-turn movements should be considered as a "starting-point." Left-turn phasing should only be provided if it will improve operations or safety. The guidelines shown in Figure 3-5 can be used to make this determination. They can also be used to determine whether the left-turn phase should operate as protected-plus-permitted or protected-only. These guidelines were derived from guidance provided by Kell and Fullerton (*32*) and by Orcutt (*28*).

Full application of Figure 3-5 requires knowledge of the crash history, intersection geometry, sight-distance, left-turn delay, and volume conditions. The following assumptions can be made if such information is unavailable:

1. If collision frequency is unavailable, it should be assumed to be less than the critical number.
2. If the road is built to satisfy AASHTO criteria, as described in the *Green Book* (*5*), then it should be assumed that sight distance is available.
3. If the cycle length is unknown, it should be assumed to be 60 s in duration (this assumption can be checked using guidance provided in a subsequent section).

The flowchart in Figure 3-5 has two alternative paths following the check of left-turn volume. One path requires knowledge of left-turn delay; the other requires knowledge of the left-turn and opposing through volumes. The left-turn delay referred to is that delay incurred when no left-turn phase is provided. If no information is available on the left-turn delay, the alternative path based on volume should be evaluated.

The application of Figure 3-5 requires the separate evaluation of each left-turn movement. This evaluation should consider both the morning and afternoon peak demand hours of the average day. Left-turn phasing should only be provided for the left-turn movement that satisfies the guidelines in Figure 3-5. For the purposes of the engineering study, if left-turn phasing is needed for one peak hour, then it should be provided for the entire day.



Critical Number of Collisions C_c
On one approach, C_c = 4 left-turn collisions per 1 year or 6 left-turn collisions per 2 years.
On both approaches, C_c = 6 left-turn collisions per 1 year or 10 left-turn collisions per 2 years.

Variables
V_{lt} = left-turn volume on the subject approach, veh/h
V_o = through plus right-turn volume opposing the subject left-turn movement, veh/h

Figure 3-5. Guidelines for determining the potential need for a separate left-turn phase.

Common Sequences. This section describes common phase sequences used when some type of left-turn phase protection is provided. One of the more commonly used sequences serves both left-turn movements during the first phase (i.e., a "leading" left-turn phase) and both through movements during the second phase. This sequence tends to pair movements having similar volume together. The advantage of such a pairing is that it provides some efficiency by concurrently serving movements that have similar volumes and service time needs. Phase Sequence B in Figure 3-4 illustrates this approach for the north-south road.

Modern, dual-ring controllers improve on the aforementioned practice by allowing for the concurrent service of several movements, including the adjacent left-turn and through movements. This type of service allows the left-turn and the through movement to be served in an overlapping manner. This capability improves intersection efficiency when opposing left-turn pairs or opposing through movement pairs have dissimilar volumes and is particularly effective when implemented with actuated control. Phase Sequences C and D in Figure 3-4 illustrate this approach for the north-south road.

Both leading and lagging left-turn phases have advantages and disadvantages. Kell and Fullerton (*32*) list these advantages and disadvantages, many of which relate to driver expectancy and behavior during the transition between the left and through phases. Local practice will often dictate the consistent use of a leading or lagging left-turn phase sequence throughout the jurisdiction.

There is one noteworthy disadvantage associated with a lagging, protected-plus-permitted left-turn phase. This disadvantage relates to the potential conflict between left-turn vehicles clearing the intersection at the end of the through phase and the opposing through vehicles. This problem is sometimes referred to as the "left-turn trap" and is described more fully by Orcutt (*28, p. 27*). The trap can be avoided by forcing the simultaneous termination of the leading through phases or by eliminating the permitted portion of the protected-plus-permitted left-turn operation. The problem can also be avoided by using leading left-turn phasing.

If local practice does not dictate the order of presentation of the left-turn phases, it should be assumed that the left-turn phase (if needed) leads the adjacent through phase. The adequacy of this assumption can be checked during Step 3 of the engineering study and a lagging left-turn phase can be used if it is found to yield better operation.

Special Sequences. As noted in the preceding section, typical practice is to serve opposing left-turns during one phase and opposing through movements during a different phase. However, there is occasionally the need to provide a separate phase for each approach. This need generally occurs when the travel paths of opposing traffic movements physically overlap within the conflict area. When a separate phase is provided for each approach, the phase sequence is often referred to as "direction separation," "split phasing," or "sequentially phased roads."

Direction separation phasing is less efficient than other phase sequences when the left-turn and adjacent through volumes are not equal. As these volumes are rarely equal, direction separation phasing is rarely efficient and should be avoided. When possible, intersection geometry should be modified so that left-turn movements can be served simultaneously during a common phase.

Basic Controller Settings

The development of an efficient timing plan for the signal phase sequence is an essential task when designing the traffic signal alternative for the engineering study. The components of this plan vary depending on (1) whether a pretimed, semi-actuated, or full-actuated control mode is used and (2) whether an isolated or coordinated operation is used. A pretimed timing plan includes specification of cycle length and a green interval duration for each phase. A full-actuated timing plan includes specification of a minimum green, a maximum green, and a unit-extension setting for each phase. A semi-actuated plan for a coordinated system includes a system cycle length, force-off and yield points, and a reference point offset. It also includes minimum green, maximum green, and unit-extension settings for the actuated (non-coordinated) phases.

Despite the aforementioned differences among control modes, there are some basic settings that all modes have in common. Specifically, they all require yellow and all-red interval durations for each phase. They also require some type of minimum green interval. A procedure is described in this section for determining these interval settings, as they apply to both the pretimed, semi-actuated, and full-actuated control modes.

Two sections follow this section (i.e., Controller Settings for Isolated Operation and Controller Settings for Coordinated Operation). These sections describe procedures for determining controller settings specific to the method of signal operation. Within each of these sections, separate subsections describe procedures for determining settings for the pretimed and actuated control modes.

The procedures described in this section and the two that follow are intended to provide a balance between the effort needed to develop a reasonable timing plan and the accuracy needed for the engineering study. Other timing plans are possible and can often be defined using other procedures or software analysis tools. Regardless of method used, the analyst's goal here is to develop a reasonably efficient timing plan that fairly represents the traffic signal alternative in the engineering study. This procedure is not intended for use in the final design of a timing plan.

Number of Timing Plans. An efficient timing plan should be one that results in minimal delay to all vehicles entering the intersection throughout the day. To achieve this result, the analyst needs to establish a timing plan that accommodates variability in traffic demands throughout the day. The nature of this accommodation will vary depending on whether the control mode is pretimed or actuated.

For pretimed control, the analyst should develop no more timing plans than would typically be implemented on a time-of-day basis in his (or her) jurisdiction. Typically, this would mean determining the green interval durations for one to three timing plans, using either software-automated or manual techniques. Each plan would be based on volumes representative of the applicable period. For example, Plan 1 could be based on the peak hour of flow during the morning and would be used for the 2-hr period from 7:00 to 9:00 a.m. Plan 2 could be based on the afternoon peak and apply to the 2-hr period from 4:00 to 6:00 p.m. Finally, Plan 3 could be based on a representative off-peak period (or weekend day) and apply to all non-peak hours.

Actuated control is better able to adapt to demand variability than is pretimed control. For isolated operation, one well-constructed timing plan should be able to serve traffic demands for the entire day. Typically, this would mean determining the maximum green settings suitable for both the morning and afternoon peak demand hours. For a given phase, the larger of the two maximum green settings determined in this manner would be set in the controller and used for the entire day. For non-isolated operation, the cycle length, force-off, yield-point, and offset settings would need to be determined for both peak hours and for a representative off-peak period.

Change Interval Duration. Fundamental components of the timing plan are the yellow warning and all-red clearance intervals. Together, these two intervals represent the change interval. Kell and Fullerton (*32*) indicate that there is considerable diversity in practice regarding the duration of the change interval. Although they discuss several commonly used strategies, they do not explicitly recommend any one strategy. They do offer a table of change interval durations that are based on theoretic relationships of driver response and deceleration. The table offered by Kell and Fullerton is reproduced as Table 3-6. Unless local practice dictates otherwise, it is recommended that the analyst use the change interval durations listed in this table for the engineering study.

Table 3-6 also provides minimum phase durations based on pedestrian crossing times. Guidance for using this portion of the table is provided in the next section.

Minimum Green Time. The minimum green duration of a phase is dependent on driver expectancy and, in some situations, pedestrian crossing time. Driver expectancy should be accommodated in the minimum green for all phases (left-turn and through). In contrast, pedestrian crossing needs should only be accommodated in through movement phases that have some pedestrian demand but no pedestrian call button. In general, all phases should have a minimum duration of 7.0 to 10.0 s to satisfy the expectancy of waiting drivers. Minimum green durations based on pedestrian crossing times generally exceed this amount and vary widely, depending on the width of the road being crossed.

For the purpose of the engineering study, it is recommended that a minimum green duration of 8.0 s be used for all phases, unless local practice dictates other values. The one exception to this rule applies to through phases that cannot be activated by a pedestrian call button. These phases should be long enough for pedestrians (traveling in the same direction as the vehicles served) to cross the intersecting road. The minimum pedestrian phase duration P_p would equal the combined pedestrian reaction time and crossing time. The minimum green duration G_m would then equal the larger of 8.0 s or the minimum phase duration *less* the change interval Y (i.e., G_m = larger of: 8.0 or $P_p - Y$). Typical minimum pedestrian phase durations are listed in the last row of Table 3-6. These durations are based on reaction times and walking speeds reported by Kell and Fullerton (*32*).

Controller Settings for Isolated Operation

This section describes a procedure for determining the controller settings for isolated intersections. The procedure

TABLE 3-6 Change interval duration and pedestrian-based phase duration

Approach Speed, km/h	Yellow Warning Interval, s	Width of Intersection, m				
		10	15	20	25	30
		Change Interval [1] (yellow warning plus all-red clearance intervals) (Y), s				
30	3.0	4.3	4.9	5.5	6.1	6.7
40	3.0	4.3	4.8	5.2	5.7	6.1
50	3.3	4.5	4.8	5.2	5.6	5.9
60	3.8	4.7	5.0	5.3	5.6	5.9
70	4.2	5.1	5.3	5.6	5.8	6.1
80	4.7	5.4	5.7	5.9	6.1	6.3
90	5.2	5.8	6.0	6.2	6.4	6.6
Min. Ped. Phase Duration [2] (P_p), s		14	18	22	27	31
Approach Speed, mph	**Yellow Warning Interval, s**	**Width of Intersection, feet**				
		30	50	70	90	110
		Change Interval [1] (yellow warning plus all-red clearance intervals) (Y), s				
20	3.0	4.2	4.9	5.5	6.2	6.9
25	3.0	4.2	4.7	5.3	5.8	6.4
30	3.2	4.3	4.8	5.2	5.7	6.2
35	3.6	4.5	4.9	5.3	5.7	6.1
40	3.9	4.8	5.1	5.5	5.8	6.1
45	4.3	5.1	5.4	5.7	6.0	6.3
50	4.7	5.3	5.6	5.9	6.2	6.4
55	5.0	5.7	5.9	6.1	6.4	6.6
Min. Ped. Phase Duration [2] (P_p), s		13	18	23	28	33

Notes:

1 - Based on the following equation: $Y = t + V/(2\ a) + (W + L)/V$ where, Y = change interval, t = driver perception-reaction time (= 1.0 s), a = deceleration rate (= 3.0 m/s^2, 10 ft/s^2), W = width of intersection, V = approach speed, and L = length of vehicle (= 6.1 m, 20 ft).

2 - Based on the following equation: $P_p = t_p + (W - w_l)/\ V_p$ where, P_p = minimum phase duration for pedestrians, t_p = pedestrian reaction time (= 7.0 s), W = width of intersection, w_l = one-half the width of the last traffic lane crossed (= 1.5 m, 5 ft), and V_p = 15th percentile walking speed of pedestrians (= 1.2 m/s, 4 ft/s).

has been developed for application to intersections with both pretimed and actuated control modes. Prior to discussing the procedure, there is a brief discussion of cycle length, as it relates to pretimed and actuated intersection operation. Then, a seven-step procedure is described for determining the settings for pretimed control. Finally, a procedure is described for determining the settings for actuated control. This latter procedure is developed as a three-step extension of the pretimed procedure. As such, it requires completion of the first six steps of the pretimed procedure.

Cycle Length. The efficient operation of a pretimed, signalized intersection is highly dependent on cycle length. Webster (*33*) demonstrated that overly long and short cycle lengths increase delay. He derived a relationship between the minimum delay cycle length and critical lane volume. This relationship is shown in Figure 3-6. The trends in this figure are based on an effective saturation flow rate of 1,500 veh/h/ln and 4.0 s lost time per phase. This saturation flow rate reflects the demand peaks, lane use, and traffic mix found at a typical intersection.

As the trends in Figure 3-6 indicate, low to moderate volumes can yield minimum-delay cycle lengths less than 40 s. In contrast, high volumes can yield cycle lengths in excess of 180 s. When the cycle length is exceptionally short or long, the analyst may wish to consider modifying the number of lanes on one or more approaches (i.e., reducing the number of lanes if the cycle length is below 40 s and increasing their number if the cycle length exceeds 180 s).

For actuated operation, a well-designed detection/control system will yield minimal delay operation by ensuring optimally utilized phase durations and cycle lengths. Therefore, for the engineering study of an actuated traffic signal alternative, it can be assumed that the cycle length obtained from Figure 3-6 is reasonably close to the average cycle length achieved by actuated operation for the analysis period. As described in a subsequent section, Actuated Phase Settings and Detection Design, this average cycle length can be used to determine reasonable maximum green settings when the proposed intersection has actuated control.

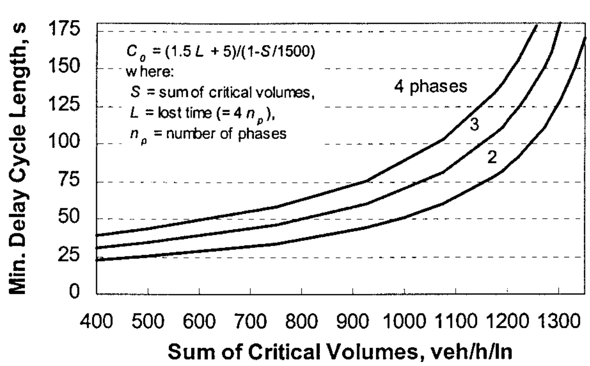


Figure 3-6. Minimum delay cycle length.

Pretimed Phase Interval Durations. Appendix B of Chapter 16 in the *Highway Capacity Manual 2000* (*15*) describes a procedure for determining the green interval duration of a pretimed phase. A simplified version of this procedure is described in Tables 3-7 and 3-8. These tables can be used to determine reasonably efficient green interval durations that should be adequate for the engineering study process. Many software analysis tools are also available that can automate the process of determining pretimed phase duration. Such tools can be used instead of the manual process described in this section.

The procedure described in this section is based on the following assumptions: (1) the left-turn phases lead the through phases, (2) one or more left-turn lanes are provided when a left-turn phase is used, and (3) both left-turn movements on a given road are protected when a left-turn phase is used. Other procedures should be considered when these assumptions are not appropriate.

The procedure for determining the green interval duration is described in terms of an example application. Blank versions of the worksheets are provided in Appendix C. The example intersection geometry and demand volumes (in veh/h) are illustrated in Figure 3-7. The east-west road is proposed to have a protected-plus-permitted left-turn phase. The north-south road will not have protected left-turn phasing. The through phases will need to be long enough to serve pedestrians. The speed on the north-south road approaches is 40 km/h (25 mph); that on the east-west road approaches is 60 km/h (40 mph).

Step 1. Input Volume and Lane Geometry. The procedure for defining a pretimed timing plan starts with Table 3-7. This worksheet is used to determine the total critical lane volume. This volume is then used in Table 3-8 to determine an appropriate cycle length and phase durations. The worksheet is completed from top to bottom. Initially, the movement volumes and lane counts are entered in the Volume and Lane Geometry Input section of the worksheet. For this analysis, the right-turn volume is combined with the through movement volume. Similarly, any exclusive right-turn lanes would be included in the count of through lanes for a given approach.

Step 2. Compute Adjusted Movement Volumes. The Phase Sequence Section of the table is completed next. This section is divided into three parts, depending on the type of left-turn phasing provided. Each road is considered separately. The first part of this section is completed when a road does not have a protected left-turn phase. The second part is completed when a road has a protected-plus-permitted left-turn phase. The third part is completed if protected-only left-turn phasing is used. The first and second parts require the computation of adjusted volumes to account for the effect of permitted left-turn activity. This computation is omitted in the third part, because protected-only left-turn phasing is used.

TABLE 3-7 Critical volume worksheet

CRITICAL VOLUME WORKSHEET								
General Information								
Location: *Pretimed Intersection*					**Analysis Period:** ______ to ______			
Volume and Lane Geometry Input								
Approach:	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Movement, No.: [1]	LT, 5	TH+RT, 2	LT, 1	TH+RT, 6	LT, 3	TH+RT, 8	LT, 7	TH+RT, 4
Volume (v_i), veh/h i = 1, 2, 3, ... 8	*105*	*502*	*201*	*806*	*93*	*408*	*57*	*104*
Lanes (n_i)	*1*	*2*	*1*	*2*	*0*	*2*	*0*	*1*
Phase Sequence	**1 Phase** (protected through & permitted left)				**1 Phase** (protected through & permitted left)			
Opposing Volume ($v_{o,i}$), veh/h	v_6 =		v_2 =		v_4 = *104*		v_8 = *408*	
LT equivalence ($E_{L,i}$) (Fig. 3-8)		1.0		1.0	*1.5*	1.0	*2.1*	1.0
Sneakers (S_i), veh/h	90	0.0	90	0.0	90	0.0	90	0.0
Adjusted volume (v^*_i) [= E_L (v_i - S_i) $\geq$ 0.0]					*5*	*408*	*0*	*104*
Lane volume ($v_{n,i}$) [= v^*_i/ n_i]{see note 2}					*0*	*206*	*0*	*104*
Critical volumes (v_c), veh/h/ln	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) =				Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) = *206*			
	2 Phase (with protected-plus-permitted left)				**2 Phase** (with protected-plus-permitted left)			
Permitted capacity ($c_{p,i}$), veh/h	60	0.0	60	0.0	60	0.0	60	0.0
Adjusted volume (v^*_i) [= (v_i - $c_{p,i}$) $\geq$ 0.0]	*45*	*502*	*141*	*806*				
Lane volume ($v_{n,i}$) [= v^*_i/ n_i], veh/h/ln	*45*	*251*	*141*	*403*				
Critical volumes (v_c) {see note 3}, veh/h/ln	Larger of: ($v_{n,1}$, $v_{n,5}$)= *141*		Larger of: ($v_{n,2}$, $v_{n,6}$)= *403*		Larger of: ($v_{n,3}$, $v_{n,7}$)=		Larger of: ($v_{n,4}$, $v_{n,8}$)=	
	2 Phase (with protected-only left)				**2 Phase** (with protected-only left)			
Lane volume ($v_{n,i}$) [= v_i/ n_i], veh/h/ln								
Critical volumes (v_c) {see note 3}, veh/h/ln	Larger of: ($v_{n,1}$, $v_{n,5}$)=		Larger of: ($v_{n,2}$, $v_{n,6}$)=		Larger of: ($v_{n,3}$, $v_{n,7}$)=		Larger of: ($v_{n,4}$, $v_{n,8}$)=	

Notes:

1 - Numbers follow NEMA movement-based phase numbering system (LT = left-turn, TH = through, RT = right-turn).

2 - If there is no left-turn lane for a given approach, then the lane volume for the left-turn movement equals 0.0 and the lane volume for the through movement is based on the total, adjusted approach volume. For example, if the eastbound approach has no left-turn lane (i.e., n_5 = 0), then $v_{n,5}$ = 0.0 and $v_{n,2} = (v^*_5 + v^*_2)/ n_2$.

3 - Critical volume for protected left-turn phases is based on the following assumptions: (1) left-turn phases lead adjacent through phases, (2) one or more exclusive left-turn lanes exist, and (3) both left-turn movements are protected.

For this example, the first part of the Phase Sequence Section is completed for the north-south road because it does not have left-turn phasing. The opposing volumes for both the north and southbound left-turn movements are entered in the first row in the Phase Sequence Section. This information is combined with that in the next three rows to convert the left-turn volume into an equivalent through volume. The left-turn equivalence factor E_L is obtained from Figure 3-8. Values for this factor are based on information provided in Appendix C of Chapter 16 in the *Highway Capacity Manual 2000* (*15*).

The northbound left-turn movement shares a lane with the northbound through movement and is opposed by 104 southbound vehicles per hour. Figure 3-8 indicates that these conditions result in a left-turn equivalency factor of 1.5. This factor is used, along with the number of vehicles that clear at the end of the through phase (i.e., sneakers), to determine the

TABLE 3-8 Controller setting worksheet with calculations for pretimed control

CONTROLLER SETTING WORKSHEET								
General Information								
Location: *Pretimed Intersection*					**Analysis Period:** ______ to _______			
Change Interval and Minimum Green								
Approach:	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Movement, No.: [1]	LT, 5	TH+RT,2	LT, 1	TH+RT,6	LT, 3	TH+RT,8	LT, 7	TH+RT,4
Yellow + all-red (Y_i), s (Table 3-6) {see note 2}	Y_2 = ***5***	***5***	Y_6 = ***5***	***5***	Y_8 = --	***5***	Y_4 = --	***5***
Ped. phase time ($P_{p,i}$), s (Table 3-6) {see note 3}	0.0	***15***	0.0	***15***	0.0	***21***	0.0	***21***
Minimum green ($G_{m,i}$), s [= larger of: ($P_{p,i}$-Y_i, 8.0)]	***8***	***10***	***8***	***10***	--	***16***	--	***16***
Critical Volume Summary {see note 4}								
1 phase E-W (n_p = 2) 1 phase N-S	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) =				Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) =			
1 phase E-W (n_p = 3) 2 phases N-S	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) =				Larger of: ($v_{n,3}$,$v_{n,7}$)=		Larger of: ($v_{n,4}$,$v_{n,8}$)=	
2 phases E-W (n_p = 3) 1 phase N-S	Larger of: ($v_{n,1}$,$v_{n,5}$)= ***141***		Larger of: ($v_{n,2}$,$v_{n,6}$)= ***403***		Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) = ***206***			
2 phases E-W (n_p = 4) 2 phases N-S	Larger of: ($v_{n,1}$,$v_{n,5}$)=		Larger of: ($v_{n,2}$,$v_{n,6}$)=		Larger of: ($v_{n,3}$,$v_{n,7}$)=		Larger of: ($v_{n,4}$,$v_{n,8}$)=	
Sum of critical volumes (Σv_c), veh/h/ln	Σv_c = ***750*** No. phases (n_p) = ***3***				Min. Delay Cycle (C_o) = ***46*** (Fig. 3-6) Cycle Length (C) = ***75***			
Pretimed Phases	Pretimed (or non-actuated) phase time				Pretimed (or non-actuated) phase time			
Critical volumes by phase ($v_{c,i}$), veh/h/ln {see note 5}	***141***	***403***	***141***	***403***	--	***206***	--	***206***
Green duration (G), s [= $v_{c,i}/\Sigma v_c$ (C - 4n_p) + 4 -Y_i]	***11***	***33***	***11***	***33***	--	***16***	--	***16***
Actuated Phases	Actuated phase maximum green setting				Actuated phase maximum green setting			
Lane volume ($v_{n,i}$), veh/h/ln (from Critical Vol. Wksht.)								
Min.-delay green ($G_{o,i}$) s [= $v_{n,i}$/ Σv_c (C_o - 4n_p) + 4 -Y_i]								
Maximum green setting, s [= larger of:($G_{m,i}$+12,1.3$G_{o,i}$)]								
Unit extension, s (Table 3-10)								

Notes:

1 - Numbers follow NEMA movement-based phase numbering system (LT = left-turn, TH = through, RT = right-turn). Complete the columns for the left-turn movements (i = 1, 3, 5, 7) *only* if a corresponding left-turn phase exists.

2 - Compute the change interval ($Y + AR$) for the through phases only. If a left-turn phase exists then set its change interval equal to that associated with the adjacent through movement.

3 - If pedestrians are served on a through phase that does not have pedestrian detection (i.e., no ped. button or ped. signal) then use Table 3-6 to determine the minimum pedestrian phase time; otherwise, use P_p = 0.0 s.

4 - Obtain from the Critical Volume Worksheet the critical volume that is associated with each movement. Only one phase combination (or row) should be used.

5 - Record the critical phase volume in all cells that correspond to the movements served.

"adjusted" left-turn volume. It is suggested that 90 sneakers per hour be used for this analysis. For the northbound left-turn movement, the adjusted volume is computed as 5 veh/h (= 1.5[93 – 90]). For through movements, the adjusted volume is equal to the actual through movement volume (no adjustment is needed).

The adjusted volume for the east-west road is based on the calculations described in the second part of the Phase Sequence Section. The adjusted left-turn volume is based on a conservative estimate of the permitted left-turn capacity. It is suggested that 60 veh/h be used for this analysis. For the eastbound left-turn movement, the adjusted volume is

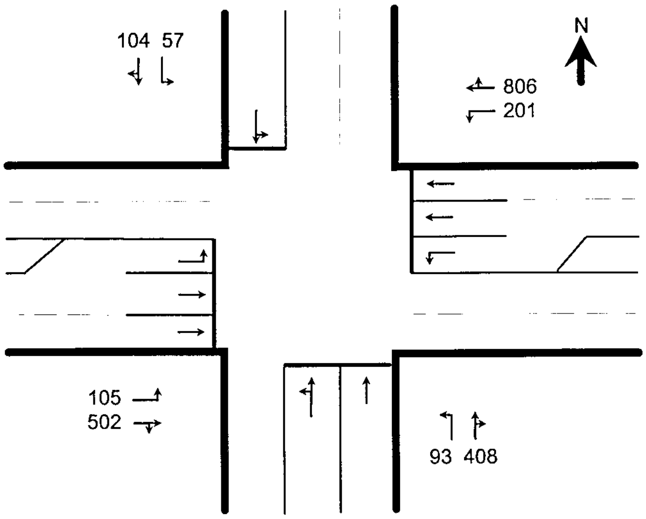


Figure 3-7. Example intersection used to illustrate timing plan development.

computed as 45 veh/h (= 105 − 60). For through movements, the adjusted volume is equal to the actual through movement volume.

Step 3. Compute Lane Volumes. For this step, the adjusted volume is used to compute the lane volume for each movement. For movements with exclusive lanes, the lane volume is computed as the adjusted volume divided by the number of lanes. However, as indicated in Footnote 2 to Table 3-7, the lane volume for a shared-lane approach is computed as the total, adjusted approach volume divided by the number of through lanes.

The north and southbound approaches both have shared lanes; therefore, the provision in Footnote 2 applies to each approach. The lane volume for the northbound approach is computed as 206 veh/h/ln (= [5 + 408]/2).

The east and westbound approaches both have exclusive lanes; therefore, the lane volumes are computed as the adjusted

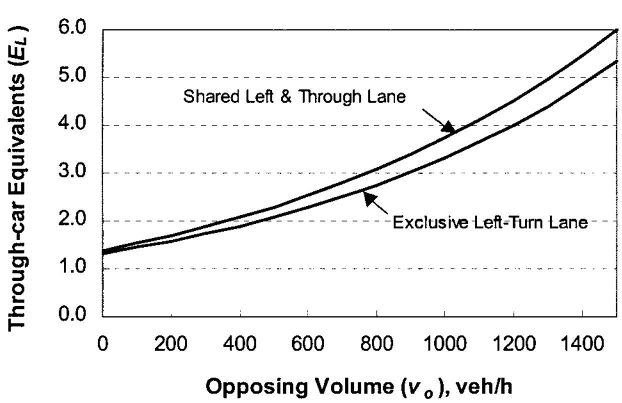


Figure 3-8. Through-vehicle equivalents for permitted left-turn vehicles.

volume divided by the number of lanes. The lane volume for the eastbound through movement is 251 veh/h/ln (= 502/2).

Step 4. Compute Critical Volumes. As a final step on the Critical Volume Worksheet, the critical lane volumes are identified. A critical volume represents the largest lane volume served by a given phase. The rules for making this identification are listed in the rows labeled Critical Volumes.

Based on the rules listed in the worksheet, the critical volume for the north-south road represents the largest of the lane volumes on the north and southbound approaches. Thus, the critical lane volume is 206 veh/h/ln for the north-south road.

Two volumes are required for the east-west road because there are two phases being used to serve east-west traffic. One volume represents the larger lane volume of the two left-turn movements; the second volume represents the larger volume of the two through movements. Thus, the critical volume pair for the east-west road is 141 and 403 veh/h/ln.

Step 5. Identify Change Interval and Minimum Green Duration. To complete the development of the timing plan, the Controller Setting Worksheet (i.e., Table 3-8) must be completed. The columns that correspond to through traffic movements should always be completed. Those columns that correspond to left-turn movements should only be completed when the movement is served by a left-turn phase.

As a first step, Table 3-6 is consulted to determine the change interval duration and the pedestrian phase time for the through movements. The left-turn phase change interval can be assumed to equal that associated with the adjacent through movement. It can also be assumed that there is no pedestrian service during a left-turn phase (i.e., $P_p = 0.0$). Finally, the minimum green duration is computed as the larger of the time needed to satisfy driver expectancy (i.e., 8.0 s) and the time needed to serve pedestrians (if appropriate).

For the north-south through movement, Table 3-6 indicates that a change interval of 5 s is appropriate for an approach speed of 40 km/h (25 mph) and a crossing distance of 18 m (60 ft) [based on five lanes at 3.6 m (12 ft) each]. Table 3-6 also indicates that a pedestrian phase time of 21 s is needed for the 18-m (59-ft) crossing distance. This phase time translates into a minimum green time of 16 s.

Step 6. Compute Critical Volume Sum and Cycle Length. The critical volume sum and cycle length are determined by completing the Critical Volume Summary Section of Table 3-8. This section is completed by transferring the critical volumes from the Critical Volume Worksheet, computing their sum, and then using this sum to determine the minimum-delay cycle length C_o. The minimum-delay cycle length is obtained from Figure 3-6 for a given critical volume sum and number of phases.

For the example intersection, the particular cells completed in the Critical Volume Worksheet are a reminder that two phases are used to serve east-west traffic and one phase to serve north-south traffic. This phase combination is represented by the row labeled “2 phases E-W, 1 phase N-S.” The corresponding critical volumes are transferred to this row from the Critical Volume Worksheet. The sum of these three critical volumes is found to be 750 veh/h/ln. Consultation of Figure 3-6 indicates that the minimum-delay cycle length for this volume (and three-phase operation) is 46 s.

Step 7. Compute Green Interval Duration. The green interval duration for each phase is determined by completing the Pretimed Phases Section of the worksheet. In the first row of this section, the critical phase volume from the preceding section is transcribed to **all** cells corresponding to the movements served by the phase. Finally, these volumes are used to compute the green interval duration for each phase. The equation used for this purpose is:

$$G_i = \frac{v_{c,i}}{\sum v_c}(C - 4n_p) + 4 - Y_i \tag{1}$$

where

G_i = green interval duration for movement i (i = 1, 2, 3, . . . , 8), s;
$v_{c,\,i}$ = critical volume for movement i, veh/h/ln;
Σv_c = sum of critical volumes, veh/h/ln;
C = cycle length, s
n_p = number of phases, and
Y_i = change interval for movement i, s.

Equation 1 distributes the available green time to the various movements in proportion to the critical volume of the corresponding phase. It assumes a phase lost time of 4.0 s. The green intervals are based on a cycle length that is initially set to equal the minimum-delay cycle length C_o. If one or more of the resulting green intervals do not satisfy the corresponding minimum green duration, then the cycle length should be increased and Equation 1 reapplied. This process is repeated until all green intervals exceed the required minimums.

For the north-south road, the critical volume of 206 veh/h/ln is recorded in both the northbound and southbound columns. Then, the north-south green interval duration is computed as 8 s (= 206/750 ∗ [46 − 12] + 4 − 5). Unfortunately, this duration is less than the minimum green duration of 16 s. To resolve this deficiency, the cycle length is increased and all green intervals are recalculated (using Equation 1). This process is repeated until the green interval durations for each movement just satisfy the minimum time needed. For this example, a cycle length of 75 s is found to yield green intervals that satisfy the corresponding minimums. It should be noted that a cycle length that is different from the minimum-delay cycle length will likely produce longer delays; however, this is an unavoidable consequence of satisfying the minimum green requirement.

Actuated Phase Settings and Detection Design. Chapter 11 of the *Manual of Traffic Signal Design* (*MTSD*) (*32*) describes procedures for determining the minimum green, maximum green, and unit extension settings for an actuated movement. The guidance provided in the *MTSD* for the minimum green setting is consistent with that provided in a previous section, Minimum Green Time. The guidance provided for the maximum green setting is based on the use of the minimum-delay pretimed green interval duration. Specifically, the *MTSD* suggests that the maximum green setting for a movement can be estimated by computing the minimum-delay green interval and then multiplying it by a factor ranging from 1.25 to 1.50.

Table 3-9 illustrates the recommended procedure for using the Controller Setting Worksheet to determine reasonable actuated controller settings for the engineering study. This procedure requires that Steps 1 through 6 from the previous section are completed first. Then, Steps 7 though 9 from in this section would be completed. As noted previously in the section, Cycle Length, the cycle length obtained from Figure 3-6 is assumed to equal the average cycle length achieved by actuated operation.

The example intersection used in the preceding section is also used in this section to demonstrate the procedure steps. The pedestrian provisions noted previously for the “pretimed” example intersection are modified for this demonstration. Specifically, pedestrian buttons are provided for all approaches; therefore, pedestrian considerations will not affect the through phase duration.

Step 7. Compute Minimum-Delay Green Intervals. As a first step in determining the maximum green setting, the lane volumes are transferred from the Critical Volume Worksheet to the first row in the Actuated Phases Section of the Controller Setting Worksheet. These volumes are then used to compute the minimum-delay green interval for each actuated movement. This green interval is computed from Equation 1 with the cycle length equal to the value obtained from Figure 3-6 (i.e., $C = C_o$).

For the example intersection, the lane volume of 403 veh/h/ln for the westbound through movement yields a minimum-delay green interval of 17 s (= 403/750 [46 − 12] + 4 − 5).

Step 8. Determine Maximum Green Setting. The maximum green setting for each movement is based on consideration of the minimum-delay green interval and the minimum green setting. Two values are computed. One value is equal to the minimum-delay green interval multiplied by a factor of 1.3. The second value is equal to the minimum green setting plus 12 s. The maximum green setting is set to equal the larger of these two values.

For the westbound through movement, the maximum green setting is found to be 22 s which is the larger of 22 (= 1.3 ∗ 17) and 20 (= 8 + 12). Following this approach, the maximum green setting for each of the other movements is found to equal 20 s.

TABLE 3-9 Controller setting worksheet with calculations for actuated control

CONTROLLER SETTING WORKSHEET								
General Information								
Location: *Actuated Intersection*					**Analysis Period:** ______ to ______			
Change Interval and Minimum Green								
Approach:	Eastbound		Westbound		Northbound		Southbound	
Movement, No.: [1]	LT, 5	TH+RT,2	LT, 1	TH+RT,6	LT, 3	TH+RT,8	LT, 7	TH+RT,4
Yellow + all-red (Y_i), s (Table 3-6) {see note 2}	Y_2 = ***5***	***5***	Y_6 = ***5***	***5***	Y_8 = --	***5***	Y_4 = --	***5***
Ped. phase time ($P_{p,i}$), s (Table 3-6) {see note 3}	0.0	***0***	0.0	***0***	0.0	***0***	0.0	***0***
Minimum green ($G_{m,i}$), s [= larger of: ($P_{p,i}$-Y_i, 8.0)]	***8***	***8***	***8***	***8***	--	***8***	--	***8***
Critical Volume Summary {see note 4}								
1 phase E-W (n_p = 2) 1 phase N-S	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) =				Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) =			
1 phase E-W (n_p = 3) 2 phases N-S	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) =				Larger of: ($v_{n,3}$,$v_{n,7}$)=		Larger of: ($v_{n,4}$,$v_{n,8}$)=	
2 phases E-W (n_p = 3) 1 phase N-S	Larger of: ($v_{n,1}$,$v_{n,5}$)= ***141***		Larger of: ($v_{n,2}$,$v_{n,6}$)= ***403***		Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) = ***206***			
2 phases E-W (n_p = 4) 2 phases N-S	Larger of: ($v_{n,1}$,$v_{n,5}$)=		Larger of: ($v_{n,2}$,$v_{n,6}$)=		Larger of: ($v_{n,3}$,$v_{n,7}$)=		Larger of: ($v_{n,4}$,$v_{n,8}$)=	
Sum of critical volumes ($\Sigma\ v_c$), veh/h/ln	$\Sigma\ v_c$ = ***750*** No. phases (n_p) = ***3***				Min. Delay Cycle (C_o) = ***46*** (Fig. 3-6) Cycle Length (C) = ***46***			
Pretimed Phases	Pretimed (or non-actuated) phase time				Pretimed (or non-actuated) phase time			
Critical volumes by phase ($v_{c,i}$), veh/h/ln {see note 5}								
Green duration (G), s [= $v_{c,i}/\Sigma v_c\ (C - 4n_p) + 4 - Y_i$]								
Actuated Phases	Actuated phase maximum green setting				Actuated phase maximum green setting			
Lane volume ($v_{n,i}$), veh/h/ln (from Critical Vol. Wksht.)	***45***	***251***	***141***	***403***	--	***206***	--	***104***
Min.-delay green ($G_{o,i}$) s [= $v_{n,i}/\ \Sigma v_c\ (C_o - 4n_p) + 4 - Y_i$]	***1***	***10***	***5***	***17***	--	***8***	--	***4***
Maximum green setting, s [= larger of:($G_{m,i}$+12,1.3$G_{o,i}$)]	***20***	***20***	***20***	***22***	--	***20***	--	***20***
Unit extension, s (Table 3-10)	***1.8***	***1.8***	***1.8***	***1.8***	--	***1.2***	--	***1.2***

Notes:

1 - Numbers follow NEMA movement-based phase numbering system (LT = left-turn, TH = through, RT = right-turn). Complete the columns for the left-turn movements (i = 1, 3, 5, 7) *only* if a corresponding left-turn phase exists.

2 - Compute the change interval ($Y + AR$) for the through phases only. If a left-turn phase exists then set its change interval equal to that associated with the adjacent through movement.

3 - If pedestrians are served on a through phase that does not have pedestrian detection (i.e., no ped. button or ped. signal) then use Table 3-6 to determine the minimum pedestrian phase time; otherwise, use P_p = 0.0 s.

4 - Obtain from the Critical Volume Worksheet the critical volume that is associated with each movement. Only one phase combination (or row) should be used.

5 - Record the critical phase volume in all cells that correspond to the movements served.

Step 9. Define the Detector Design and Unit Extension. The unit extension setting is based on the detector location and operation. It is desirable that local practice regarding detector location and operation be consulted at this point because of the numerous combinations possible. If information on local practice is not available, the following guidelines can be used to define detector locations, operation, and unit extension settings that are consistent with the level of detail needed for the engineering study.

Two detector design options are described in the remainder of this section for the purpose of determining a reasonable detector location, operation, and unit extension setting. One design option applies to (1) low-speed [i.e., 70 km/h (45 mph) or less] through movements and (2) protected left-turn movements. The other option applies to high-speed through movements. Both options assume presence-mode detector operation. It should also be noted that both options have a maximum allowable headway of 3.0 to 3.5 s. Headways in excess of this maximum would need to occur before a phase could gap-out. This maximum headway is consistent with that of the "typical" detection designs described in the *MTSD* (*32*).

For low-speed through movements and protected left-turn movements, it is suggested that a continuous stop line detection area (comprising one long loop or several 2-m (6-ft) loop detectors) be used. This detection area would be 15 m (50 ft) in length. The unit extension would range in value from 0.5 to 1.9 s, depending on approach speed. Unit extension values for typical approach speeds are listed in Table 3-10. For the example intersection, the north-south road's 40 km/h (25 mph) approach speed would require a 1.2-s unit extension. Similarly, the east-west road's 60 km/h (40 mph) approach speed would require a 1.8-s unit extension.

For high-speed through phases, it is suggested that two advance loop detectors and a stop line detection area be used for the engineering study evaluation. Both advance detections are 2 m (6 ft) in length. One advance detector is located at a point 4.5 s travel time from the stop line. A second advance loop is located 3.0-s travel time from the stop line. The equivalent travel distances are listed in Table 3-10 for a range of approach speeds. The stop line detection area is 15 m (50 ft) in length and uses the call-delay feature to effectively disable the detector's operation during green, after the initial queue clears. The call delay setting for the stop line detection area is listed in Table 3-10. The unit extension for all high-speed designs is 1.5 s.

Controller Settings for Coordinated Operation

When a traffic signal is proposed to be added to an arterial or network street system, it should be coordinated with the upstream and downstream signals. When possible, coordination should be provided for both travel directions; however, it may not be possible to achieve two-way coordination for some combinations of signal spacing, cycle length, and traffic speed.

The components of the coordinated timing plan vary depending on whether a pretimed or semi-actuated control mode is used; however, all modes include the yellow and all-red intervals for each phase. A pretimed timing plan includes defining the cycle length, reference phase offset, and green interval duration for each phase. A semi-actuated plan includes defining the system cycle length, force-off, and yield points. It also includes minimum green, maximum green, and unit extension settings for the actuated (non-coordinated) phases.

This section describes a reasonable procedure for establishing a coordinated timing plan at a level of detail suitable for the engineering study. This procedure can be used if information on local practice regarding signal coordination is not available. Although this procedure represents a "manually applied" process, it is anticipated that the analyst will prefer to automate the coordinated timing plan development process to the extent possible by selecting a software analysis tool (in Step 2, Select Analysis Tool) that has this capability.

The procedure described in this section extends the material described in the previous two sections (Basic Controller Settings and Controller Settings for Isolated Operation). As such, the worksheets completed in the previous section will need to be completed for the subject intersection when it

TABLE 3-10 Unit extension settings for suggested detector designs[1]

Design	Approach Speed, km/h (mph)	Unit Extension, s	Call-Delay Setting, s	Distance Between Detector and Stop Line, m (ft)	
				1st Upstream Detector	2nd Upstream Detector
Low-speed	30 (20)	0.5	n.a.	n.a.	n.a.
	40 (25)	1.2	n.a.	n.a.	n.a.
	50 (30)	1.5	n.a.	n.a.	n.a.
	60 (40)	1.8	n.a.	n.a.	n.a.
	70 (45)	1.9	n.a.	n.a.	n.a.
High-speed	70[a] (45)	1.5	1.7	88 (297)	58 (198)
	80 (50)	1.5	1.6	100 (330)	67 (220)
	90 (55)	1.5	1.5	113 (363)	75 (242)
	100 (60)	1.5	1.4	125 (396)	83 (264)

Notes:
n.a. - not applicable.
1 - Based on (1) presence mode detection, (2) two 2-m (6-ft) advance loops at 4.5 and 3.0 s travel time, (3) 15-m (50-ft) stop line detection area, (4) for speeds in excess of 70 km/h (45 mph) and above, call-delay is invoked for stop line detection during green, and (5) non-locking memory. Designs yield a maximum allowable headway in the range of 3.0 to 3.5 s.
a - High-speed design applies to approach speeds in excess of 70 km/h (45 mph). Values for 70 km/h (45 mph) are shown to facilitate interpolation.

is part of a coordinated system. This section describes only coordination-related extensions to the material described in the previous sections.

Cycle Length and Offset. The cycle length used for the subject intersection C should be set equal to the existing signal system cycle length C_s. This cycle length should be compared with the minimum delay cycle length C_o (identified in Step 6). If C_s is less than 0.75 C_o, then the subject intersection's geometry may need modification (e.g., lanes added) to lower C_o. The goal is to reduce the subject intersection's minimum-delay cycle length until the existing system cycle length exceeds 75 percent of C_o.

The term "relative" offset is used in this section. It represents the time between the start of through green at two adjacent intersections (one of which would be the subject intersection). The relative offset needed to provide two-way progression through the proposed signal can be estimated as one of two values. Either it will be about 0.5 C or it will be about 0.0 s. Both of these offsets should be evaluated in terms of the platoon arrival times from the two upstream intersections. The preferred offset will be the one that has vehicles arriving on green in both directions. As an alternative, one-way progression can be provided in the peak travel direction by using a relative offset equal to the travel time.

The remainder of this section describes a technique for selecting a system cycle length and a relative offset. This technique can be used when there is some flexibility in setting the system cycle length. It is applicable to the following two cases:

1. The major road currently has only one signalized intersection in the vicinity of the proposed signal, or
2. The existing system cycle length can be changed to promote good two-way progression through the proposed signalized intersection.

Each of these cases is discussed separately in the paragraphs that follow.

The cycle-selection technique is based on the use of either a "single alternate" or "double alternate" progression scheme. Each scheme has a unique type of offset relationship between adjacent intersections. The single alternate scheme uses a relative offset that equals the travel time between intersections. The double alternate scheme uses a relative offset equal to 0.0 s between one pair of intersections and a relative offset equal to the travel time between the next pair of intersections.

Both schemes are based on the assumption that the through green interval and the cycle length are the same at each intersection. However, small differences in green interval duration do not preclude the use of these techniques. Another assumption inherent to these schemes is that the distance (or spacing) is constant between intersection pairs. Both schemes are

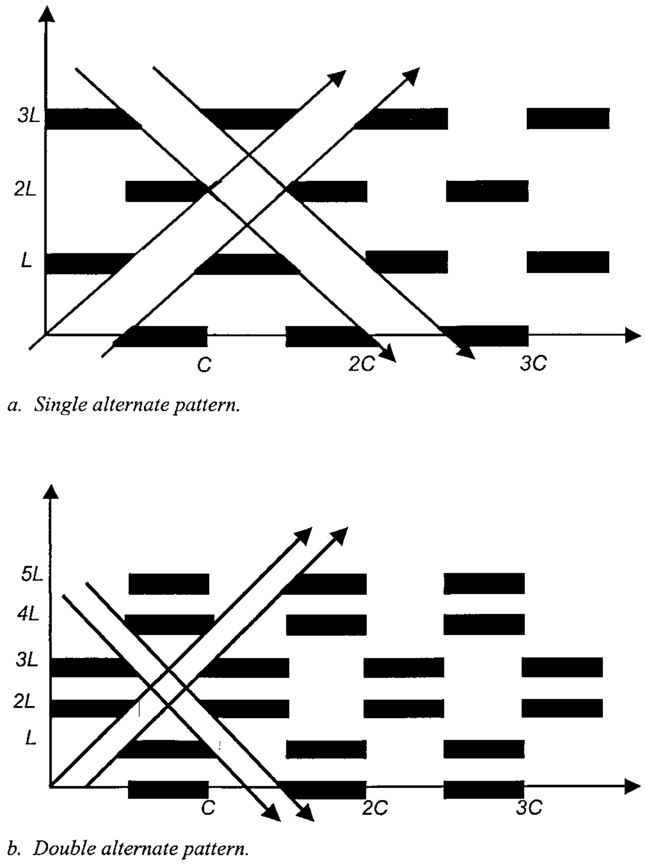
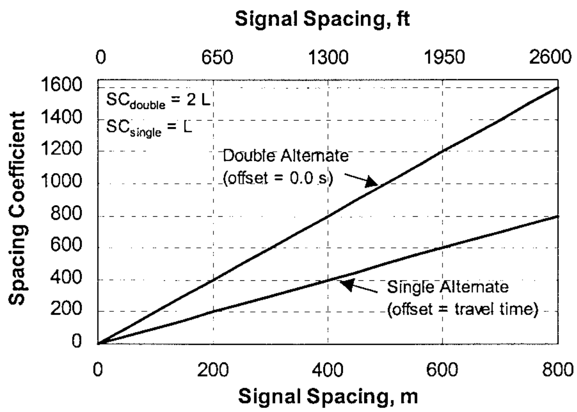


Figure 3-9. Two offset relationships that provide good two-way progression.

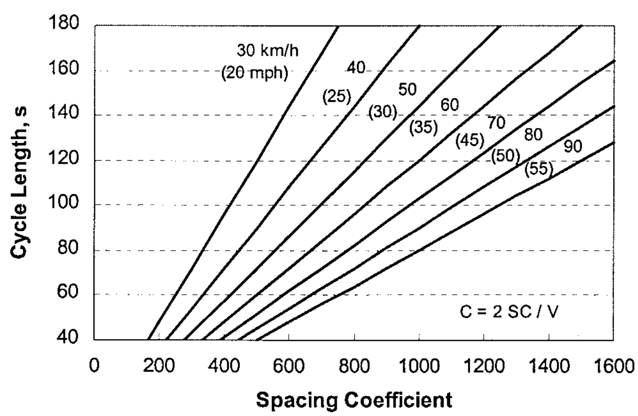
shown in Figure 3-9 in terms of a series of intersections spaced L units apart with each intersection operating at a cycle length of C s.

As the trends in Figure 3-9 indicate, the single alternate scheme is more efficient than the double alternate scheme because the single alternate scheme has a progression band that is equal to the full width of the green interval. The band for the double alternate scheme is less than the width of the green interval; the amount of reduction depends on the travel time between the intersections. In general, the double alternate scheme provides reasonably good progression when the green-to-cycle-length ratio G/C for the major-road through phase is 0.5 or more.

Case 1. Only One Signalized Intersection in the Vicinity of the Proposed Signal. As a first step, the distance between the existing and proposed intersections must be identified. Then, this distance is converted into a spacing coefficient SC using the "single alternate" trend line in Figure 3-10a. Finally, this coefficient is used with Figure 3-10b to determine the cycle length for a given average running speed on the major road. The relative offset between the through movement phases at the two intersections would be equal to the travel time.



a. Spacing coefficient.



b. Cycle length.

Figure 3-10. Relationship between spacing, progression scheme, speed, and cycle length.

If the cycle length obtained by the aforementioned technique is unacceptably short, the process should be repeated using the double alternate trend line. In this situation, the relative offset between the through phases at the two intersections would be equal to 0.0 s.

To illustrate the Case 1 technique, consider that a new signalized intersection is proposed to be located 400 m (1,300 ft) from a nearby signalized intersection. No other signals exist within 1.5 km (4,900 ft). The major-road running speed is 70 km/h (45 mph). Figure 3-10a indicates that the spacing coefficient is 400 for the single alternate progression scheme. Figure 3-10b indicates that a spacing coefficient of 400 and a speed of 70 km/h (45 mph) require a cycle length of 40 s. The offset would equal the travel time of 20 s. For this example, 40 s is determined to be too short for a system cycle length. Thus, Figure 3-10a is revisited to find that the double alternate scheme will yield a coefficient of 800. A check of Figure 3-10b indicates that this coefficient, when combined with the 70 km/h speed, yields a more reasonable cycle length of 80 s. The offset would equal 0.0 s. As noted previously, the double alternate scheme is most effective when the *G/C* ratio is 0.5 or more for all through phases.

Case 2. Existing System Cycle Length Can Be Changed to Promote Progression. The first step is to identify the distance between the existing signalized intersections. If this distance is not constant among intersection pairs, then the distance between the two signalized intersections that bound the proposed signal should be used. This distance is then converted into a “spacing coefficient” using the “single alternate” trend line in Figure 3-10a. Next, this coefficient is used with Figure 3-10b to determine the ideal cycle length for a given average running speed on the major road. As for Case 1, if the cycle length is unacceptably short, then the double alternate scheme should be used to determine the cycle length. Regardless of the scheme selected, the cycle length obtained from Figure 3-10b should be used when the proposed signal is installed, the relative offset to the proposed signal would equal 0.0 s, and the *G/C* ratio for the through phases should be 0.5 or more.

One exception to the technique described in the previous paragraphs exists when the distance between existing intersections is 800 m (2,600 ft) or more and the proposed signal is to be located at a point midway between existing intersections. In this special case, a single alternate scheme will generally provide very good two-way progression. To determine the proper cycle length, the spacing coefficient is defined to equal the distance between the proposed signal and an existing signal. This coefficient would then be used with Figure 3-10b to determine the ideal cycle length for a given average running speed on the major road. The relative offset would equal the travel time.

To illustrate the Case 2 technique, consider that a new signalized intersection is proposed to be located midway between two existing signalized intersections that are spaced 800 m apart (2,600 ft). The major-road running speed is 70 km/h (45 mph). Figure 3-10a indicates that the spacing coefficient is 800 for the single alternate progression scheme. Figure 3-10b indicates that a spacing coefficient of 800 and a speed of 70 km/h (45 mph) require a cycle length of 80 s This cycle length is acceptable, so the proposed intersection should be designed to operate with a cycle length of 80 s. In addition, all through movement phases should have a *G/C* ratio of 0.5 or more to ensure reasonable progression bandwidth. Finally, the relative offset to the proposed intersection should be 0.0 s.

The example intersection in the preceding paragraph is also a candidate for consideration of the aforementioned exception to Case 2. The distance between the proposed and existing signals would equal 400 m (1,300 ft). This distance equates to a spacing coefficient of 400. Figure 3-10b indicates that this coefficient corresponds to a 40-s cycle length for a speed of 70 km/h (45 mph). The offset would equal the travel time of 20 s. The use of a single alternate scheme is attractive because it provides very good progression; however, it can only be used for this example if the intersections can operate effectively at a relatively short, 40-s cycle length.

Pretimed Phase Interval Durations. If the intersections in the signal system are pretimed, the procedure described in the section, Controller Settings for Isolated Operation, can be used to develop the timing plan. Specifically, the procedure described for pretimed intersections can be used to determine the appropriate phase interval durations. One exception to this procedure relates to the cycle length used in the Controller Setting Worksheet (i.e., Table 3-8). The cycle length used for all calculations should equal either (1) the existing system cycle length or (2) the cycle length obtained from the procedure described in a previous section, Cycle Length and Offset. The cycle length selected by either means would then be used to compute the pretimed phase durations in the worksheet.

Actuated Phase Settings and Detection Design. If the proposed intersection will use semi-actuated control in a coordinated arterial system, the procedure described in a previous section (Controller Settings for Isolated Operation) should be used to determine the appropriate phase settings and minor movement detection design. Specifically, the analyst should complete the Controller Setting Worksheet (i.e., Table 3-9) using the Actuated Phases section to compute the maximum green settings for the non-coordinated phases. The cycle length used for all calculations should equal either (1) the existing system cycle length or (2) the cycle length obtained from the procedure described in a previous section, Cycle Length and Offset.

Detectors should be used for all non-coordinated phases (i.e., all left-turn and minor-road through phases). If local practice on detection design is not known, the "low-speed" design described previously should be used. Specifically, this design would consist of a continuous stop line detection area (comprising one long loop or several 2-m loop detectors). The detection area would be 15 m (50 ft) in length. The unit extension would range in value from 0.5 to 1.9 s, depending on approach speed. Unit extension values for typical approach speeds are listed in Table 3-10.

Force-Off and Yield-Point Settings. Semi-actuated coordinated systems require specification of force-off and yield point settings to ensure smooth traffic progression. These settings represent fixed points during the cycle and are referenced to the start of the major-road through phase at the "master" intersection (assumed to be the existing upstream intersection for the purpose of the engineering study). To determine these settings, the analyst should complete the Controller Setting Worksheet (i.e., Table 3-8) using the Pretimed Phases section to compute equivalent pretimed durations for all signal phases.

The yield-point *YP* setting defines when the major-road through phase can be terminated (so as to serve waiting minor-movement vehicles) without disrupting progression. The yield-point value can be set to equal the relative offset plus the equivalent pretimed green duration of the major-road through movement phase (i.e., $YP_{2,6}$ = Offset + $G_{2,6}$). This relationship is shown in Figure 3-11 for three-phase operation with the major-road through phase denoted by movement numbers 2 and 6.

The force-off settings FO_i define when each minor-movement phase should be terminated so that all phases can be served and green can be returned to the major-road through phase in time to serve an arriving platoon. One force-off setting is computed for each minor-movement phase. The force-off setting for the minor phase that follows the major-road through phase is computed as the sum of the yield-point, major-road change interval, and minor-phase equivalent pretimed green duration (i.e., $FO_1 = YP_{2,6} + Y_{2,6} + G_{4,8}$). If a second minor phase exists, then its force-off would be computed as the sum of the previous phase's force-off setting, its change interval, and the second minor-phase equivalent pretimed green duration (i.e., $FO_2 = FO_1 + Y_{4,8} + G_{1,5}$). This process would repeat for all additional minor-movement phases.

Example. For example, consider the example intersection previously considered in the development of Tables 3-7 and 3-8. This intersection has a leading, protected left-turn phase

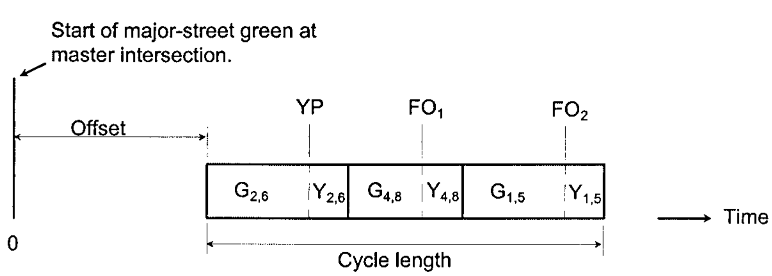


Figure 3-11. Relationship between offset, cycle length, yield-point, force-off settings.

on the east-west road. Assume that a 75-s cycle length with a 32-s relative offset was determined to provide good two-way progression. Based on this assumption, the equivalent green durations shown in Table 3-8 can be used to determine the yield-point and force-off settings. The yield point for the east-west through phase can be computed as 65 s (= 32 + 33). The next phase to occur in the phase sequence is the north-south phase. The force-off for this phase is computed as 86 s (= 65 + 5 + 16). As this value exceeds the cycle length, one increment of the 75-s cycle length is subtracted to obtain a "relative" force-off setting of 11 s. The last phase to occur is the east-west left-turn phase. The force-off setting for this phase is computed as 27 s (= 11 + 5 + 11).

Guidelines for Use of Stochastic Simulation Models

Overview

This section describes guidelines for the use of stochastic simulation models. These models recreate the random events that define an individual vehicle's journey through the simulated street network. Although the simulation of random events is very realistic, it results in an element of uncertainty in the output measures of effectiveness (MOEs). This uncertainty stems from the fact that the sequence of random events changes from one simulation run to the next (assuming a different random number seed is used each run) and produces slightly different MOE values at the end of each run. This section provides guidance as to how the variability in MOEs can be minimized to the extent that reasonable confidence can be obtained in the conclusions reached from the engineering study.

Random Arrivals

By definition, the headway between arriving vehicles at an isolated intersection is a random variable that is exponentially distributed. An accurate assessment of isolated intersection delay requires that the simulation model replicate this behavior. The analyst should verify that arrival headways on the isolated intersection approach vary with some degree of randomness. Some clue as to the variability of arrivals can be found by observing the vehicle headways in the graphic animation package that accompanies the simulation model. However, a model's ability to generate random headways should be verified by consultation of its user's manual.

Concerns about random arrival headways are also relevant to the evaluation of systems of signals. Specifically, arrival headways to the external approaches of intersections located along the arterial street (or around the network) should follow the Negative Exponential distribution. External approaches are those approaches that do not have an upstream signalized intersection.

Random Number Seed

Most simulation models use one user-specified, random number seed to determine the sequence of simulated random events. The analyst must change this seed with each new simulation run to ensure that the random processes in the system are fully reflected in the output MOEs. If the simulation model uses different seed numbers to control different aspects of the simulated events, then all seeds should be changed with each new simulation run.

Several techniques can be used with simulation models to minimize or isolate the variability due to some random events. For example, one simulation model allows the analyst to specify that the same drivers, vehicles, and routes are used for each simulation run while still allowing driver decisions to be randomly selected. This approach helps to reduce the simulation time needed to identify whether significant differences in delay exist among alternative control strategies. However, it may also bias the estimate of a movement's true delay when several runs are made and the results averaged because only a subset of the driver population will be represented in the averaged values. In short, variance-reduction techniques can be useful *if* they are properly applied. Their proper application requires an understanding of the underlying statistical issues and the questions for which variance-reduction can be helpful in answering. If there is any uncertainty about the use of variance-reduction, it is recommended that the guidance offered in the previous paragraph be followed.

Minimum Simulation Initialization Time

The initialization time used for the simulation relates to the period of time that elapses (relative to the start of the simulation run) before the model starts to collect MOE statistics. In general, the first few vehicles to enter the simulated network do not experience the level of delay and interaction that is experienced by subsequent vehicles. Thus, these initial vehicles may bias the MOE statistics if they are included in the MOE calculations.

To minimize the potential bias resulting from "start-up" effects, it is suggested that the analyst set the initialization time to a value that exceeds the travel time between the two most-distant points in the simulated network. This approach is intended to allow the simulated street network time to "fill" with vehicles before statistics are collected. Some stochastic simulation models automatically define (and apply) an initialization period based on the aforementioned rule.

Simulation Period and Run Duration

As mentioned previously, the variability in the predicted MOEs is a result of randomness in the simulated system. This variability effectively reduces the precision of the predicted values. However, a desired precision can be achieved by increasing the number of observations included in the average MOEs reported by the simulation model. Several techniques are available for this purpose; however, the simplest and most direct technique is to increase the simulation period until the desired precision is achieved.

In this guide, the "simulation period" is defined as the total time a given evaluation period is simulated, as measured by the "clock" in the simulation software. The simulation period may exceed the period represented by the evaluation period (e.g., morning peak hour). The next three sections describe a procedure for determining the simulation period. This procedure is based on three assumptions: (1) that delay is the basis for evaluating the acceptability of an alternative; (2) that the delays to individual traffic movements will be compared with a threshold value [e.g., a specified delay level-of-service value obtained from the *Highway Capacity Manual 2000* (*15*)]; and (3) that the overall intersection delays will be compared among alternatives with the lower delay alternative being given preference.

Run Duration. The simulation period should consist of one or more "runs," with each run having a 1-hr duration. Thus, if the evaluation period is the morning peak hour and it is determined that a 2-hr simulation period is needed to achieve the desired precision, then the simulation period should consist of two runs of 1-hr duration each. The desired MOEs would be computed as the average of the individual MOEs from each run.

The benefits of using a 1-hr run duration are that (1) it is consistent with the evaluation period used in the engineering study and (2) it provides a common time basis for comparison when one or more movements are oversaturated. With regard to this second point, a fixed run duration facilitates the comparison of delay among alternatives because it accounts for the time dependency associated with oversaturated movement queues (when they exist).

Other approaches to defining run duration are possible. For example, simulating volume patterns that occur during several, consecutive 1-hr periods would provide a better estimate of delay when an intermediate hour has one or more oversaturated movements. However, this approach would require considerably more analysis effort than needed for the engineering study. It is believed that the use of a common, 1-hr duration for each simulation run provides the best balance between the effort required for the engineering study and the precision needed in the analysis results.

Minimum Simulation Period Based on Individual Movement Delay. One condition used to determine if a proposed alternative is "effective" is that each traffic movement has an acceptable level of service. This verification requires the comparison of the predicted movement delay with a specified threshold delay value. In this situation, it is important to know if the predicted delay is truly larger than the threshold value and that any difference is not due only to random variation. The ability to make this determination can be enhanced by providing a sample size that is large enough to minimize random influences.

Figure 3-12 can be used to determine the simulation period needed to limit the uncertainty in the delay estimate to 10 percent or less. The 10-percent trend line reflects a 90-percent confidence level that the true delay is no more than 10 percent larger than the model-predicted delay. The predicted delay can be inflated by 10 percent and the result compared with the specified threshold delay value. Statements can then be made about whether the threshold value has likely been exceeded. The development of Figure 3-12 is described elsewhere by Bonneson and Fontaine (*20*).

A 10-percent error limit is recommended for the engineering study. Error limits other than 10 percent can be obtained by multiplying the time obtained from Figure 3-12 by the factor: $f = (10 / \text{new error percentage})^2$. For example, a 20-percent error limit would require a simulation period that is one-fourth the duration of that needed for a 10-percent error limit.

Minimum Simulation Period Based on Average Intersection Delay. The comparison of delays between one or more alternative intersection improvements is also an important consideration in the engineering study. Specifically, it is important to know that the predicted average intersection delay for one alternative is truly less (or more) than that predicted for another alternative and that the difference is not due to random variation. The ability to make this determination can be enhanced by providing a sample size that is large enough to minimize random influences. For this discussion, an existing intersection is also considered as an alternative (i.e., the "do-nothing" alternative).

Figure 3-13 can be used to define the minimum simulation period needed to determine when one alternative is truly operating with less overall delay than another alternative.

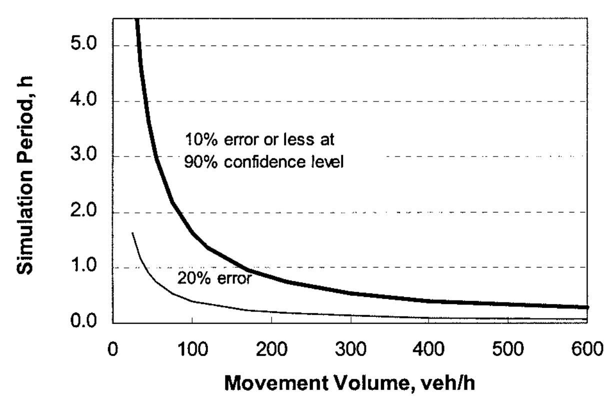


Figure 3-12. Minimum simulation period when comparing delay to a threshold value.

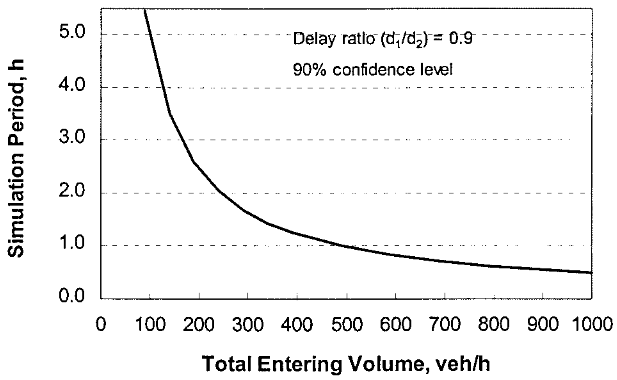


Figure 3-13. Minimum simulation period when comparing delays between two alternatives.

The difference in average intersection delay among the two alternatives is reflected in the "delay ratio" term, which is the ratio of the smaller delay to the larger delay. The trend line indicates the simulation run time needed to have 90 percent confidence that the difference between a pair of predicted delays is significant. The development of Figure 3-13 is described elsewhere by Bonneson and Fontaine (*20*).

Procedure. The procedure for determining the time elements of the simulation process includes consideration of the minimum simulation period needed for each traffic movement and for the overall intersection. The largest of these minimums would then be used to define the total simulation time period. The minimum simulation period is then divided into a series of individual simulation runs of 1-hr duration.

As a first step, Figure 3-12 is consulted for each traffic movement of interest for a given alternative. Only traffic movements that have exclusive lanes can be examined independently. Movements that share a lane should be combined and examined as a single movement. This process is repeated for each alternative.

As a second step, Figure 3-13 is consulted once for each alternative using a flow rate that corresponds to the total entering volume for the respective alternative. One simulation period is obtained from the figure for each alternative.

As a third step, the largest single time period obtained from the first and second steps is used to define the minimum simulation period used. This period is then rounded to the nearest number of whole hours (e.g., 1.6 h is rounded to 2.0 h). A series of 1-hr simulation runs are then completed so that the total simulation time equals the simulation period. Each run should be based on a different random number seed. The movement and intersection delays obtained from each run are then averaged to produce the desired MOEs.

Example. To illustrate the procedure, consider the intersection shown in Figure 3-14 (volumes in veh/h). The intersection is currently unsignalized; however, a traffic signal alternative is being considered. A simulation analysis is to be conducted to determine the delay associated with each alternative. The analyst wants to know if movement delays exceed a threshold delay level of 55 s/veh and if overall delay differences of 10 percent or more (i.e., a delay ratio of 0.9) are likely to be significant from a statistical standpoint.

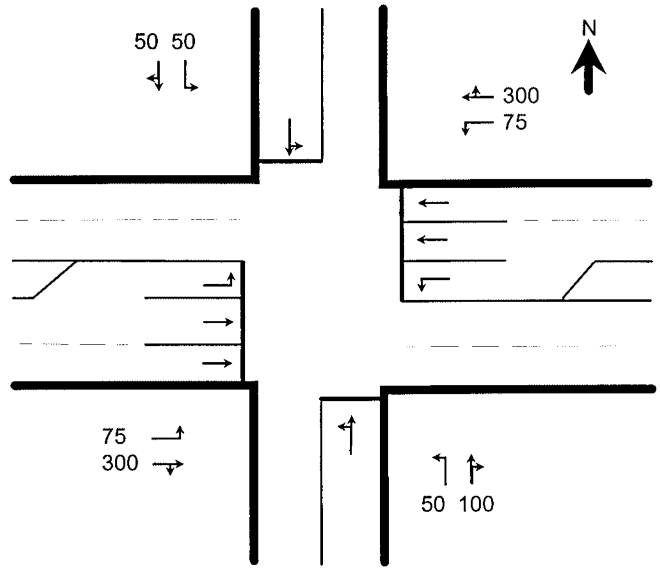


Figure 3-14. Example intersection used to illustrate simulation run control procedure.

The first step is to determine the minimum simulation period required for threshold assessment. This time period is dictated by the lowest movement volume because it will require the longest simulation time. The 50-veh/h left-turn movements on the north and south approaches share a lane with the through movements and thus should be combined with the through movements. This combination yields shared-lane volumes of 100 and 150 veh/h for the north and south legs, respectively. After reviewing all volumes, the 75-veh/h left-turn volumes on the east and west approaches are the lowest volume movement at the intersection and dictate the minimum simulation period based on movement considerations. Figure 3-12 indicates that a minimum simulation period of 2.2 hr would be needed based on individual movements.

The second step is to determine the minimum simulation period required for the comparison of alternatives. This period is dictated by the total entering volume at the intersection. This volume is 1,000 veh/h for the example intersection. Figure 3-13 indicates that a minimum simulation period of 0.5 hr will be necessary for both alternatives (as they both have an entering volume of 1,000 veh/h).

The third step is to determine the minimum simulation period for the engineering study. For this example, the individual movement delay evaluations require a simulation duration of 2.2 hr and the overall intersection delay comparisons require a simulation duration of 0.5 hr. The larger duration of these two values, 2.2 hr, dictates the simulation duration. As a final step, this duration can be rounded to 2.0 hr. In summary, the simulation period should consist of two, 1-hr simulation runs.

At the conclusion of the two, 1-hr runs for one alternative, predicted delays of 51 and 45 s/veh are obtained for the eastbound left-turn movement. These two delays are averaged to obtain an average delay of 48 s/veh. This delay, combined with a 10-percent error limit, would indicate that the true mean delay is less than 53 s/veh. Because 53 s/veh is less than the threshold of 55 s/veh, the analyst can report (with 90-percent confidence) that the movement delay does not exceed the threshold level. Similar statements could be made about the delays for the other traffic movements.

Average overall intersection delays for the unsignalized and signalized alternatives are computed as 50 s/veh and 40 s/veh, respectively. The corresponding delay ratio is 0.8 (= 40/50). As 0.8 is less than 0.9, the analyst can conclude (with 90-percent confidence) that the average 10-s/veh delay difference is statistically significant and that the two alternatives are associated with different delays.

CHAPTER **4**

ALTERNATIVE SELECTION

This chapter describes the alternative selection stage of the engineering assessment process. This process is used to evaluate problem intersections and identify effective improvement alternatives. During the alternative selection stage, the alternatives identified at the end of the engineering study stage are evaluated to determine their effect on traffic operations, safety, and other factors. The alternative determined to be the “best overall alternative” based on this evaluation is then recommended for implementation at the subject intersection.

PROCESS

Overview

The alternative selection stage consists of three steps. These steps, described in this chapter, are

1. Identify impacts,
2. Select the best alternative, and
3. Document the study

In the first step, each alternative that was deemed to be “effective” at the conclusion of the engineering study stage is evaluated to determine its effect on traffic operations and other factors. In the second step, these effects are aggregated and compared in order to determine the best overall alternative. Finally, in the third step, the analyst documents the study process, the relevant findings, and the recommended course of action in a study report.

The objectives of the alternative selection stage are (1) to define a rational process for selecting the best overall alternative and (2) to document the results of the assessment process in a study report. These objectives are achieved through the development and application of a uniform procedure for alternative selection and through the agency’s formal documentation of the study process. The steps involved in this stage are described in the remainder of this chapter.

Step 1. Identify Impacts

This step is intended to assist the analyst in making a comprehensive assessment of all effects that would result from the implementation of each alternative. This assessment consists of the following two tasks:

a. Identify Decision Factors
b. Assess Degree of Impact

In the first task, a list is developed that includes the decision factors that will be considered in the selection process. Then, in the second task, the engineer systematically evaluates the effect of the alternative on each factor. This evaluation is based on a quantitative assessment of the degree of effect each alternative would have on a given factor.

1-a. Identify Decision Factors

The factors considered in the alternative selection process may include traffic operations, traffic safety, construction cost, aesthetics, environment, and right-of-way requirements. The specific factors considered for a given project will depend on the conditions present at the problem location, public awareness of the problem, and size (or extent) of the proposed improvements. For small projects with little visibility, the engineer may choose to focus on the traffic operations and safety impacts. Cost factors may also be considered for moderately sized projects. For large projects or those with considerable visibility, all of the factors should be considered. In this latter situation, the analyst may need to enlist the support of other professionals to evaluate impacts to the environment or adjacent property.

Agency preferences may also dictate which factors are considered. These preferences may reflect the philosophy of the agency administrators, the amount of quantitative information needed to assess a given factor, the degree of reliance placed on engineering judgment, and the desired precision of the engineering study. Table 4-1 lists the factors that may be considered in the selection of alternative improvements at problem intersections (the arrangement shown is arbitrary and implies no order of importance).

The factors listed in Table 4-1 are consulted during this task. Those factors that have particular relevance to one or more alternatives (including the existing intersection, if applicable) are identified. Traffic operations (including capacity and level of service) should always be one of the factors considered. Engineering judgment should be used to determine if impacts to the other, non-operations-related factors are enough to warrant their consideration. The factors selected should reflect

TABLE 4-1 Factors that may be considered in the alternative selection stage

Factor	Description
Traffic Operations	Quality of service provided to motorists, pedestrians, and bicyclists.
Traffic Safety	Crash risk to motorists, pedestrians, and bicyclists; separation of modes.
	Public health and safety; timely service to emergency vehicles.
Direct Cost	Design, right-of-way, and construction (initial costs).
	Maintenance and operating (ongoing costs).
Other	Aesthetics, environment, property access, property values, economic activity in area.

conditions at the subject intersection, adjacent properties, and the surrounding street network (if applicable).

1-b. Assess Degree of Impact

During this task, the factors identified in Task 1-a are reviewed and evaluated for each alternative. The evaluation should include some assessment of the effect of an alternative on the associated factor.

The effect of each factor should be quantified in terms of a representative performance measure. For example, the operational effect of each alternative could reflect the total delay experienced during the average day. If safety is evaluated, the safety impact of each alternative could reflect the annual number of crashes expected at the site. If direct-cost is evaluated, the cost impact of each alternative could reflect its initial and annual costs. If the other factors are being considered, their effects could also reflect a quantitative performance or impact measure that has some economic basis.

The precision of the impact estimate should reflect a balance between the importance of the study and the implications of not identifying the best overall alternative. The operational effects should be estimated during the engineering study stage. If other factors are being included, their effects can be estimated using a combination of experience and engineering analysis. In this regard, an engineer who has experience estimating the effects of similar projects may be able to provide reasonable impact estimates for the alternatives of interest. Agency reports may also be a source of information about the effects of similar projects. The analyst may also consult the literature to determine the effects of some factors (e.g., safety).

An example application of the impact assessment process is provided in Table 4-2. Two alternatives are being compared. Alternative 1 is estimated to have 7 vehicle-hours of delay during the average day, 6 crashes per year, an initial cost of $400,000, and a $2,000 annual maintenance cost. The impacts of Alternative 2 are also listed. The crash frequency and direct-cost estimates are approximate and based on typical values from similar projects.

For the example illustrated in Table 4-2, the agency determined that it was appropriate to consider only traffic operations, crash frequency, and initial costs for this project. The data in this table indicate that Alternative 1 has lower delay, higher crash frequency, and higher direct cost than Alternative 2.

Step 2. Select Best Alternative

During this step, the analyst will select the best alternative based on consideration of the assessed impacts of the various alternatives. Initially, weights are selected for the individual factor impacts. Then, the best alternative is selected as the one having the lowest weighted total impact.

The weights used for each factor are estimated with the same level of precision used to estimate the factor impacts. To provide equity among the various factors considered, the weights can be based on the annualized worth of each factor; however, this is not a requirement. For example, if total motorist delay [in vehicle-hours per day (veh-h/day)] is used to quantify the

TABLE 4-2 Example procedure for assessing the degree of impact

Factor	Description	Units	Impact	
			Alt. 1	Alt. 2
Traffic Operations	Motorist delay	veh-h/day	7	29
Traffic Safety	Crash frequency	crashes/year	6	4
Direct Cost	Design, right-of-way, and construction (initial costs).	$ /1000	400	60
	Maintenance and operating (ongoing costs).	$ /1000	2	2
Other	Aesthetics, environment, property access, etc.	--	n.c.	n.c.

Notes:
n.c. = not considered for this project.

TABLE 4-3 Example procedure for alternative selection

Factor	Description	Units	Weight	Impact	
				Alt. 1	Alt. 2
Traffic Operations	Motorist delay	veh-h/day	4.5	7	29
Traffic Safety	Crash frequency	crashes/year	40	6	4
Direct Cost	Design, right-of-way, and construction (initial costs).	$ /1000	0.1	400	60
	Maintenance and operating (ongoing costs).	$ /1000	1.0	2	2
Other	Aesthetics, environment, property access, etc.	--	--	n.c.	n.c.
			Total:	314	299

Notes:
n.c. = not considered for this project.

impact on traffic operations, its weight would reflect the annual cost of time per vehicle. These weights should be estimated by an engineer experienced in estimating the impact of similar projects.

The annualized-worth basis for determining the weights is a suggestion; it is not a requirement of this procedure. Other weighting schemes can be rationalized and should reflect the policies of the responsible agency, the time available for alternative selection, and the experience of the analyst. However, once the weights are estimated, they should be used for subsequent projects to provide some degree of consistency in the recommendations.

An example application of this procedure is provided in Table 4-3. The impacts identified in Table 4-2 are used for this example. The weight used for the delay impact is based on an estimate of the annual cost of time per vehicle (in thousands

TABLE 4-4 Typical study report outline

No.	Section Heading	Description
1	Introduction	• Identify the location of the study. • Indicate why the study was requested and by whom. • Indicate if the location was the subject of a previous study.
2	Existing Conditions	• Identify roadway conditions and influence of upstream signalized intersections. • Include: street classification, street orientation (e.g., north-south), speed limit, cross-section components, pavement markings, and traffic control. • Discuss site characteristics that could adversely affect operations or safety.
3	Traffic Conditions	• Summarize the vehicular, pedestrian, and bicycle traffic volumes. • Describe the data collection procedures and the date the data were collected.
4	Crash History	• Summarize the analysis of at least three years of crash data. Specify the frequency of injury-related crashes. • Describe the procedures used to reduce the data.
5	Alt. Identification	• Identify the improvement alternatives considered. Justify inclusion if necessary. • If the signal alternative is considered, describe the warrant evaluation results and indicate which warrants were satisfied.
6	Alt. Evaluation	• Summarize the results of the alternative evaluation. • Identify the analysis tool used. • Indicate the level of service provided by each alternative.
7	Recommendations	• State recommended plan of action and justification based on findings. • Indicate if inter-agency agreements or exceptions to design standards are needed. • Provide an estimate of the cost of the recommended plan of action. • If the signal alternative is recommended, describe the proposed signal timing plan and the quality of traffic progression provided.
8	Attachments	• Area map • Condition diagram • Photo log (optional) • On-site observation report • Traffic count summary sheets (optional) • Collision diagram • Warrant worksheets (optional) • Output from analysis software or completed analysis worksheets (optional)

of dollars). It assumes that there are 300 equivalent average days per year and that the cost of time is $15 per vehicle-hour (i.e., $4.5 = 300 * 15 / 1000$). Thus, the product of this weight and the total delay yields an estimate of the annual cost of delay per year for a given alternative. Again, these weights are illustrative; the analyst should establish appropriate weights for his (or her) agency following these principles.

The weight used for crash frequency reflects an equivalent cost for the average crash (in thousands of dollars) and reflects the distribution of fatal, injury, and property damage crashes at intersections. The initial costs (in thousands of dollars) were given a weight of only 0.1 to reflect their annualization over a 15-year period. These weights were used to compute the weighted total shown in the last row of the table. This total represents the annual cost of each alternative (in thousands of dollars). The lower total for Alternative 2 indicates that it should be selected (over Alternative 1).

A formal economic analysis may also be used to determine the best alternative. This analysis has the advantage of being a defensible, quantitative means of assessing the relative worth of various alternatives by using financial return (in dollars) as a common basis of comparison. Its disadvantage is that it requires some effort to quantify the economic worth of each factor considered. A procedure for conducting an economic analysis of transportation projects is described in *A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements* (*34*).

Step 3. Document Study

During this step, the analyst documents the results of the study process in a study report. This report should describe the study process, the relevant findings, and the recommended course of action. The report should also include all supporting discussion, information, and summary worksheets. The content of the report should be readily understood by other engineers and agency administrators.

The study report represents the agency document that supports the action taken at the subject intersection. As such, this report must contain sufficient information to justify the recommended action. The typical study report outline is listed in Table 4-4. The information in this table is partially drawn from the Montana Department of Transportation's *Traffic Engineering Manual* (*35*).

As indicated by the information in Table 4-4, the study report should contain eight sections. The information gathered through the conduct of the procedures in Chapters 2 and 3 would be described in Sections 1, 2, 3, 5, and 6. An analysis of the intersection's crash history would be included in Section 4. A procedure for conducting the crash data analysis is described in the Institute of Transportation Engineer's *Manual of Transportation Engineering Studies* (*6*).

In general, the study report should be kept brief. As a rule-of-thumb, the report (excluding attachments) should be four to six pages in length with about one paragraph of discussion devoted to each of the bullet items in Table 4-4. A table may be the most efficient means of summarizing the traffic volumes and the crash history. A short cover letter may also be added to document the conveyance of the report to the appropriate agency administrator.

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APPENDIX A

CASE STUDY APPLICATIONS

References to software products in Appendix A are solely for the purpose of providing meaningful illustrations of the analysis tool selection process. No endorsement of a product is implied by its inclusion in the guide.

INTRODUCTION

The objective of this appendix is to demonstrate the procedures described in the *Engineering Study Guide* (*ESG*) through their application to several case studies. These case studies are based on real-world intersections; however, some aspects of their traffic demand or geometry have been modified to illustrate various elements of the procedures.

Four case studies are described in this appendix. Collectively, these case studies show the three stages of the engineering assessment process (i.e., alternative identification and screening, engineering study, and alternative selection). They also illustrate the ability of the process to deal with both isolated intersections and intersections within signal systems (i.e., non-isolated). The attributes of the case studies are listed in Table A-1.

The case studies were selected to demonstrate the manner in which the *ESG* could be used to identify and evaluate a range of alternatives. Collectively, the case studies consider both roundabout and conventional intersection geometry; the latter with two-way stop, multi-way stop, and traffic signal control. By coincidence, the signal alternative is considered in each case study. However, this result should not be construed to mean that the signal alternative is a required consideration in all engineering studies. It should also be noted that the signal alternative was not always found to be the best alternative.

TABLE A-1 List of case study attributes

No.	Improvement Category	Improvement Type	Isolated or Non-Isolated	Scenario
1	Minor	Geometry: bays	Isolated	Minor road delays at existing intersection improved by adding turn bays on both roads.
2	Major	Control: signal	Isolated	Minor road delays at existing intersection improved by converting to signal control.
3	Minor	Control: re-stripe	Isolated	Minor road delays at existing intersection improved by turn bay substituted for parking.
4	Major	Control: signal	Non-Isolated	Proposed intersection has adequate capacity with stop control because of gaps in vehicle platoons.

CASE STUDY 1

SYNOPSIS

This case study illustrates an application of the *Engineering Study Guide* (*ESG*) to an isolated, stop-controlled intersection where minor-road drivers are experiencing excessive delay. The HCS software package was used to evaluate the operation of the intersection and its proposed alternatives. The addition of right-turn bays on the north and westbound approaches and the addition of a left-turn bay on the southbound approach was recommended as the best method for alleviating motorist delay.

BACKGROUND

The local transportation agency has received many complaints about the intersection of County Routes 21 and 27. These complaints suggest that the delay to minor-road drivers may be excessive. Most of these drivers are traveling to and from a manufacturing plant located about ½ mile to the east of the intersection on County Road 27. Because of these complaints, an engineering assessment was undertaken to decide if improvements to the intersection were needed.

The intersection of County Routes 21 and 27 is in a rural community with approximately 11,000 residents. The intersection has three approach legs; one leg is stop-controlled. County Route 27 (CR 27) is the minor road; it is oriented in an east-west direction and has stop-control at the intersection. County Route 21 (CR 21) is the major road; it is oriented in the north-south direction. The speed limit is posted at 45 mph on both roads. All approaches have one traffic lane. Figure A-1 illustrates the intersection geometry.

STAGE 1: ALTERNATIVE IDENTIFICATION AND SCREENING

The first stage of the engineering assessment process involved diagnosing the problem at the intersection and determining potential corrective measures. Candidate alternatives were identified and given a preliminary screening to decide whether they represented viable solutions to the problem. The alternative identification and screening stage consists of the following steps:

1. Define problem and cause,
2. Select candidate alternatives, and
3. Select viable alternatives.

Stage 1: Step 1. Define Problem and Cause

Gather Information

The first step was to gather information about the intersection. This involved checking agency records and visiting

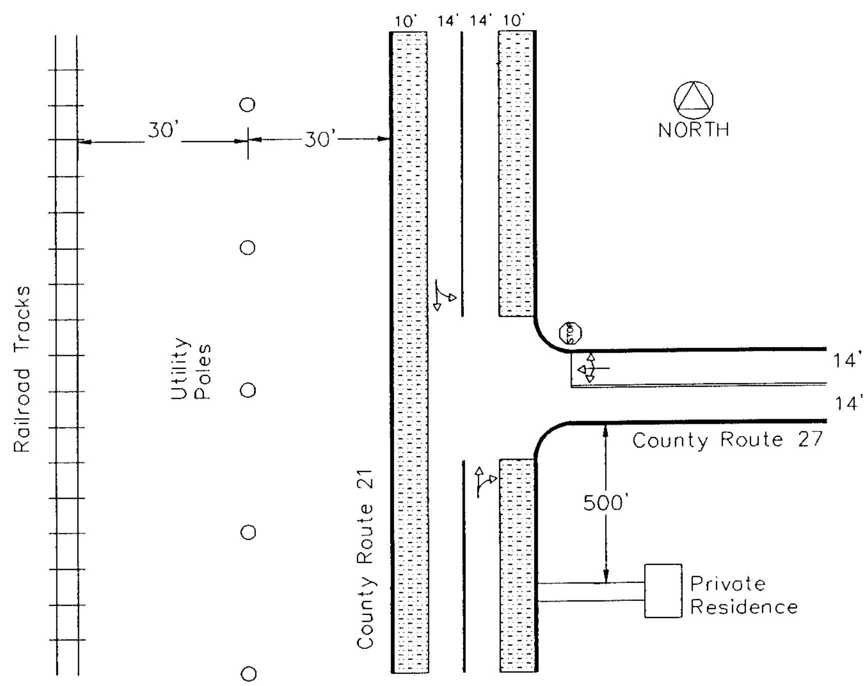


Figure A-1. Case Study 1: Existing intersection geometry (not to scale).

the site. The data gathered during this step are summarized in the remainder of this section.

Historic Data. This intersection had not been the subject of a prior engineering study, so archival traffic data were not available. Crash data were obtained from the state's Department of Public Safety. These data indicated that a total of five collisions occurred at the intersection in the last 12 months (two susceptible to correction by a traffic signal). Regional traffic counts recorded 1 year earlier indicated that the annual average daily traffic (AADT) is about 13,500 vehicles per day on CR 21 and 3,000 vehicles per day on CR 27.

Observational Study. The intersection was visited during a typical weekday to determine whether the reported delay problem was present at the intersection. The On Site Observation Report completed during this study is provided as Figure A-2. The observation study revealed that drivers on the minor road did seem to experience excessive delay during the peak hour. Much of this delay appeared to be the result of right- and left-turn vehicles having to share one traffic lane. The study also revealed that through drivers southbound on CR 21 occasionally experienced some delay and conflict because of vehicles waiting to turn left. Finally, it was noted that some conflicts occurred between northbound through and right-turn vehicles (and associated delay) when the right-turn

ON SITE OBSERVATION REPORT

LOCATION: *County Routes 21 and 27* **DATE:** *3/3/00*
CONTROL: *Stop control on County Route 27* **TIME:** *4:30 P.M.*

Isolated and Non-Isolated Intersections	**No**	**Not Sure**	**Yes**
1. Does road curvature, vegetation, buildings, parked cars, etc. block drivers' views of conflicting vehicles?	✓		
2. Is the intersection skew angle so sharp that it makes it difficult to view conflicting vehicles or complete turns?	✓		
3. Do vehicle speeds appear too high?		✓	
4. Does the delay for the minor-road right-turn appear excessive?			✓
5. Does the delay for the minor-road through appear excessive?	*n.a.*		
6. Does the delay for the minor-road left-turn appear excessive?			✓
7. Does the delay for the major-road left-turn appear excessive		✓	
8. Does the queue for the major-road left-turn ever impede major-road through traffic?		✓	
9. As major-road vehicles slow to turn, do they impede other vehicles?			✓
10. Do parking maneuvers impede other vehicles?	✓		
11. Are drivers not complying with the traffic control devices?	✓		
12. Is there evidence that one or more curb radii are too small?	✓		
13. Do pedestrians appear to cause conflict with vehicular traffic?	✓		
14. Are there guidance or control problems that could be mitigated by raised-curb channelization?	✓		
Non-Isolated Intersections			
A. Do queues from adjacent signalized intersections spillback into the subject intersection?	*n.a.*		
B. Do vehicles slowing to turn at adjacent intersections or driveways contribute to the delay to major- or minor-road drivers?	*n.a.*		
C. Is it possible that some drivers are diverting to the subject intersection because of congestion on a nearby arterial street?	*n.a.*		
D. Does the arrival pattern of major-road traffic platoons contribute to the delay to minor-road drivers?	*n.a.*		

na = not applicable.

Comments:
Large delays to minor-road drivers during peak hours.
Major road left- and right-turning vehicles sometimes obstruct through movement.

Figure A-2. Case Study 1: On site observation report.

TABLE A-2 Case Study 1: Potential problems and causes

Potential Problem	Possible Cause
Excessive delay to the minor-road movements.	• Inadequate capacity for minor-road movements. • Inadequate separation of minor-road movements.
Some delay and conflict to major-road through and left-turn movements.	• Inadequate separation of major-road movements.
Some delay and conflict to major-road through and right-turn movements.	• Inadequate separation of major-road movements.

vehicle slowed to turn right. However, the delay associated with this latter conflict appeared to be fairly small.

Site Survey. A survey was also conducted during the site visit. Key geometric features of the intersection noted during this survey are summarized in Figure A-1. Of particular note at this location are the railroad tracks, utility lines, and a nearby residence. Specifically, railroad tracks run parallel to CR 21 and are about 60 ft west of CR 21. Electrical utility poles are about one-half the way between CR 21 and the railroad tracks. A private residence is in the southeast quadrant of the intersection. A driveway to this residence is 500 ft south of the intersection on CR 21.

Define Problem and Cause

Assess Evidence. Based on the field observations, delay to minor-road drivers seemed excessive. There also appeared to be some delay and conflict associated with the major-road left- and right-turn movements. This latter problem stemmed from the interaction between the through and turning movements and the fact that these movements share a common lane on both the north and south approaches. From this evidence, it was concluded that a problem existed and that further study was justified.

Define Problem and Identify Cause. The observational study identified three potential problems at the intersection. These problems were

- Excessive delay on the minor road,
- Some delay and conflict to through and left-turn movements on CR 21 southbound, and
- Some delay and conflict to through and right-turn movements on CR 21 northbound.

Using the information provided in Table 2-1, possible causes for these problems were identified. This information is summarized in Table A-2.

Define Influence Area. The intersection is several miles from nearby signalized intersections, so it operates as an isolated intersection. The subject intersection's influence area was defined to consist of the intersection conflict area and a 300-ft length of roadway on each approach.

Stage 1: Step 2. Select Candidate Alternatives

Identify Potential Alternatives

Table 2-2 in the *ESG* was consulted to determine potential alternatives that could address the observed problems at the intersection. The alternatives identified in Table 2-2 that are applicable to the subject intersection are summarized in Table A-3.

Organize and Select Alternatives

The potential alternatives were subjected to a preliminary screening to eliminate any alternatives that would not be feasible at the site. Judgment was used to identify those alternatives that were clearly not feasible because of site-specific constraints. Based on this screening, three alternatives were eliminated. These alternatives are listed in Table A-4.

Based on this analysis, the following alternatives were found to merit further study:

TABLE A-3 Case Study 1: Potential alternatives

Possible Cause	Corrective Strategy	Potential Alternatives
Inadequate capacity for minor-road movements.	Increase approach capacity.	1. Convert to roundabout. 2. Convert to yield control. **3. Convert to traffic signal.** **4. Convert to multi-way stop control.**
Inadequate separation of minor-road movements.	Separate conflicting flows.	**1. Add second lane on minor road.** 2. Increase right-turn radius
Inadequate separation of major-road movements.	Separate conflicting flows.	**1. Add left-turn or right-turn bay.** 2. Increase right-turn radius.

TABLE A-4 Case Study 1: Alternatives eliminated

Alternative	Reason for Elimination
Convert to roundabout.	The intersection's proximity to the parallel railroad tracks and utility lines makes it unlikely that a roundabout could be built without significant cost.
Convert to yield control.	Sight obstructions in southeast quadrant (landscape shrubbery for dwelling) limit approach sight distance such that a full stop is needed on minor approach.
Increase right-turn radius.	Existing shoulders effectively provide room for large radius turns yet do not eliminate conflicts.

- Convert to traffic signal control,
- Add left-turn bay to major road,
- Add right-turn bay to major road,
- Add second lane on minor road, and
- Convert to multi-way stop control.

Stage 1: Step 3. Select Viable Alternatives

The candidate alternatives selected in Step 2 were more closely examined in this step. This examination focused on evaluating each alternative to assess its "viability" (i.e., its ability to address the observed problems effectively).

Gather Information

Identify Data. Table 2-5 in the *ESG* was used to identify the data needed to evaluate the candidate alternatives. These data are summarized in Table A-5.

Based on the information in Table A-5, the minimum amount of data that must be collected to evaluate all of the guidelines was determined. The following data were identified as necessary for guideline evaluation:

- Major- and minor-road turn movement volume (8 hours);
- Pedestrian volume (4 hours);
- Gap frequency (4 hours);
- 85th percentile approach speed;
- Progression quality;
- Minor-road delay (1 hour);
- ✓ Area population;
- ✓ Number of lanes; and
- ✓ Crash history by type.

Of these data, information about the last three items had already been gathered in a previous step. The minimum number of hours for which data would be collected is indicated in the list above.

TABLE A-5 Case Study 1: Data needed to evaluate guidelines

Data				Guideline [a, b, c] (numbers indicate hours of data needed)												
Category	Approach		Smallest Interval	Signal Warrant								MW		2L	LB	RB
	Major	Minor		1	2	3	4	5	6	7	8	B	C			
Approach volume	✔		hour	8	4	1				8	8		8	1		
		✔	hour	8	4	1				8	8		8			
Turn movement volume	✔		hour												1	1
		✔	hour											1		
Pedestrian volume	✔		hour				4	1		4			8			
Gap frequency	✔		hour				4	1								
Speed	✔		day	✔	✔	✔					✔		✔		✔	✔
Progression quality	✔		day						✔							
Delay		✔	hour			1							1			
Area population			--	✔	✔	✔					✔					
Number of lanes	✔	✔	--	✔	✔	✔				✔	✔					
Crash history by type	✔	✔	1-year							✔		✔				

Notes:

a- Guidelines:
MW - Multi-way stop control; 2L - Second lane on minor road; LB - Left-turn bay; RB - Right-turn bay.

b- Numbers at the top of the Signal Warrant column and letters at the top of the MW column refer to the warrant or criterion in *MUTCD 2000* (*A-2*).

c- Numbers shown in the table indicate the minimum number of hours for which data are collected. These hours must represent the ***highest*** volume hours.

Collect Data. Because it was unclear when the eight highest traffic hours occurred, turn movement volumes were recorded for a total of 13 hours. These counts were adjusted to represent average-day volumes using the techniques described in Appendix C. The adjusted volumes are summarized in Table A-6.

The morning peak demand occurred between 7 and 8 a.m., the afternoon peak occurred between 4 and 5 p.m., and the off-peak hour occurred between 9 and 10 a.m. No pedestrians were observed during the study. Heavy vehicles comprised about 1.0 percent of the traffic stream on each approach. A spot speed study was performed for CR 21; the 85th percentile speed was found to be 50 mph.

A stopped-delay study was performed during the afternoon peak hour (i.e., 4 to 5 p.m.). This study focused on the delay to minor-road vehicles. The average delay during the peak hour was 40 s/veh and the total delay was 2.3 veh-h.

Assess and Select Alternatives

Convert to Traffic Signal Control. The traffic signal warrants in the *Manual on Uniform Traffic Control Devices-Millennium Edition* (i.e., *MUTCD 2000*) (*A-2*) were evaluated to assess the viability of the traffic signal alternative. A complete warrant analysis for this intersection is documented in Appendix B. The following warrants were satisfied:

- Warrant 1: Eight-hour vehicular volume (Conditions A and B),
- Warrant 2: Four-hour vehicular volume, and
- Warrant 3: Peak hour.

The presence of conditions that could misdirect the signal warrant check was also assessed. A review of Table 2-15 in the *ESG* revealed that two conditions exist that could affect the warrant analysis conclusions. One condition relates to the fact that there are only three approach legs at the subject intersection. The second condition relates to the significant number of right-turn vehicles on the minor-road approach.

The discussion associated with Table 2-15 indicated that the *MUTCD's* volume-based warrants may have threshold values that are too low for a three-leg intersection. Based on the guidance in Chapter 2, the major and minor roads at a three-leg intersection may need to have volumes of 1,000 and 200 veh/h, respectively, for the eight highest hours before a traffic signal is likely to be justified. A comparison of these values with the intersection volumes indicated that the "1,000 veh/h criteria" was not met for any of the hours that turn movement volumes were collected.

The impact of minor-road right-turn volume on the accuracy of the warrant check was also investigated. Figure 2-11 in the *ESG* was consulted to determine the extent to which the right-turn volume might be reduced in the warrant check. Based on this examination, it was found that the right-turn volume probably should not be included in the warrant analysis. However, when Warrants 1 and 2 were re-evaluated without the right-turn volume, the warrants were still satisfied.

In summary, the warrant analysis indicated that a traffic signal might improve traffic operations at the intersection. However, guidelines in Chapter 2 indicated that the three-leg geometry of the intersection may affect the accuracy of the warrant check conclusions. From this analysis, it was determined that the operation of the traffic signal alternative would be carefully examined during the engineering study stage to confirm the benefits of signalization.

TABLE A-6 Case Study 1: Turn movement volumes

Hour	Volume (veh)							
	Northbound		Southbound		Major Total	Westbound		Minor Total
	Through	Right	Left	Through		Left	Right	
6 - 7 a.m.	49	36	20	248	353	35	1	36
7 - 8 a.m.	**117**	**109**	**236**	**378**	**840**	**155**	**54**	**209**
8 - 9 a.m.	117	109	78	332	636	73	21	94
9 - 10 a.m.	**99**	**109**	**87**	**253**	**548**	**94**	**34**	**128**
10 - 11 a.m.	92	91	114	260	557	132	40	172
11 a.m. - 12 p.m.	111	144	200	283	738	152	46	198
12 - 1 p.m.	181	101	54	459	795	56	13	69
1 - 2 p.m.	116	91	187	352	746	152	49	201
2 - 3 p.m.	114	132	154	359	759	140	37	177
3 - 4 p.m.	138	124	127	433	822	158	43	201
4 - 5 p.m.	**145**	**146**	**143**	**464**	**898**	**140**	**65**	**205**
5 - 6 p.m.	176	139	129	456	900	152	48	200
6 - 7 p.m.	165	83	136	434	818	164	45	209

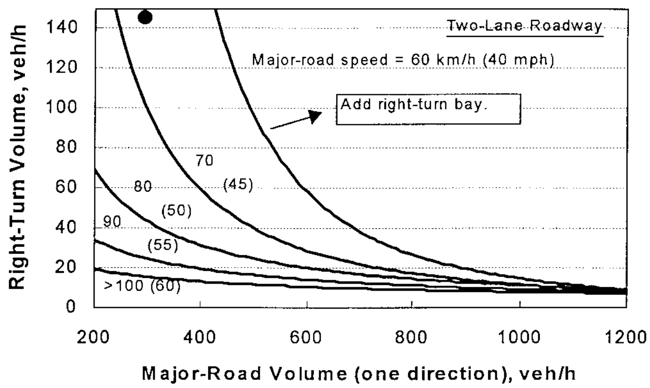


Figure A-3. Case Study 1: Check of need for right-turn bay on major road.

Add Left-Turn Bay to Major Road. Figure 2-5 in the *ESG* was examined to determine whether an exclusive left-turn bay on the major road was feasible. Based on this examination, it was determined that a left-turn bay was justified during the peak demand hour. This finding indicated that a left-turn bay on the major road would be desirable.

Add Right-Turn Bay to Major Road. Figure 2-6 in the *ESG* was examined to determine whether an exclusive right-turn bay on the major road was feasible. This figure is reproduced as Figure A-3. The northbound volume combination during the afternoon peak hour is indicated by a solid circle. Speed measurements on the major road indicated that the 85th percentile speed is 50 mph. The fact that this circle lies above the "50 mph" trend line indicates that a right-turn bay on the northbound approach is a viable improvement alternative.

Add Second Lane on Minor Road. Figure 2-4 in the *ESG* was examined to determine whether adding a second lane (i.e., an exclusive turn bay) on the minor road was feasible. This examination was based on a right-turn volume that averaged 32 percent of the approach traffic stream during the afternoon peak hour. The results of this examination are illustrated in Figure A-4.

The solid circle in Figure A-4 represents the volume occurring during the afternoon peak hour. By interpolation, the circle lies slightly above an equivalent "32%" trend line. Hence, it was concluded that an additional lane on the minor road would be beneficial. Site conditions indicated that this lane could most easily be added as a right-turn bay.

Convert to Multi-Way Stop Control. The *MUTCD* provides four criteria that can be used to determine whether multi-way stop control might improve intersection operations (these criteria are also cited in Chapter 2). One of the criteria indicates that multi-way stop control may be justified when (1) the total volume entering the intersection on the major-road approaches averages at least 210 veh/h for any 8 hours of an average day, (2) the volume of vehicles on the minor road averages at least 140 veh/h for the same 8 hours, and (3) the average minor-road delay is at least 30 s/veh during the highest hour. These volume thresholds represent "70%" levels because the major-road speed exceeds 40 mph.

Table A-7 summarizes the results of the multi-way stop criteria evaluation. Based on this examination, it was determined that the multi-way stop criteria are satisfied and that multi-way stop control is a viable alternative at this intersection.

Summary of Viable Alternative

Based on the assessment of viable alternatives, all five candidate alternatives identified in Step 2 were selected for

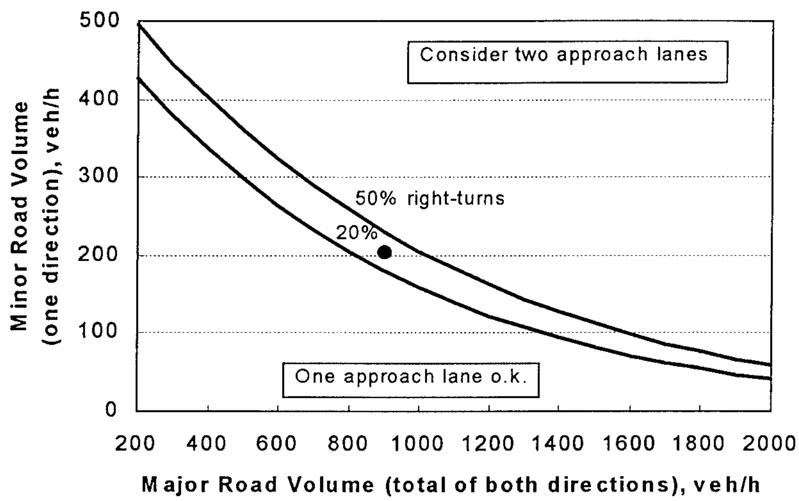


Figure A-4. Case Study 1: Check of need for second lane on minor road.

TABLE A-7 Case Study 1: Check of multi-way stop control criteria

Criterion	Minimum Level	Hours Met	Measured Delay	Satisfied?
1. Total entering volume on major road	> 210 veh/h for 8 hours	13		Yes
2. Total entering volume on minor road	> 140 veh/h for 8 hours	9		Yes
3. Average delay on minor road	> 30 s/veh for peak hour		40 s/veh	Yes

more detailed analysis in the next stage of the assessment process. These alternatives are summarized in Table A-8.

STAGE 2: ENGINEERING STUDY

The operational performance of the alternatives identified in the alternative identification and screening stage were evaluated in the engineering study stage. This stage consists of three steps:

1. Determine study type,
2. Select analysis tool, and
3. Conduct evaluation.

Stage 2: Step 1. Determine Study Type

The first step of the engineering study stage required a determination of the type of study needed. Initially, the relationship between the subject intersection and any upstream or downstream signalized intersection was evaluated. Then, the analysis detail required for the assessment of each alternative was determined.

Determine Type of Operation

The interaction between the subject intersection and any adjacent signalized intersection was evaluated to determine whether the operation of the adjacent intersection should be considered in the analysis. An investigation of the location of these intersections revealed that the nearest signalized intersection is 6 miles to the north. Figure 3-1 in the *ESG* indicates that distances exceeding 1,800 ft (when the two-way volume is 900 veh/h) are sufficient to isolate the subject intersection from the adjacent signalized intersections.

Determine Type of Evaluation

Next, the intersection was examined to determine whether a formal evaluation of alternatives was required. The *ESG* suggests that an informal evaluation (i.e., implementation and field study) is possible when only one viable alternative is applicable at an isolated intersection. However, the fact that the subject intersection is associated with several viable alternatives required the conduct of a formal evaluation.

Stage 2: Step 2. Select Analysis Tool

During this step, an analysis tool was selected for evaluating the operation of the subject intersection and its alternatives. This selection was based on an identification of the desired analysis tool capabilities and a comparison of this list with the actual capabilities of the available tools. The tool selected for use was the one that provided all of the desired capabilities.

Identify Desired Capabilities

Table 3-1 in the *ESG* was consulted to determine which analysis tool capabilities would be required for this evaluation. The minimum set of capabilities needed for the analysis tool are summarized in Table A-9.

TABLE A-8 Case Study 1: Viable alternatives

Alt.	Description	Acronym
1	Base case (existing intersection with no improvements).	TWSC-0
2	Conversion to multi-way stop control.	MWSC
3	Conversion to traffic signal control.	Signal
4	Stop control with right-turn bay on minor road.	TWSC-1
5	Stop control with right-turn bay on minor road and left-turn bay on major road.	TWSC-2
6	Stop control with right-turn bay on minor road, left-turn bay on major road, and right-turn bay on major road.	TWSC-3

TABLE A-9 Case Study 1: Desired analysis tool capabilities

Feature	Category	Capability
Traffic Control Mode	Unsignalized	Two-way stop control
		Multi-way stop control
	Signalized	Actuated signal control
Analysis Factors	Traffic Characteristics	Percentage of heavy vehicles
	Traffic Modeling	Delay to unstopped through veh.
	Geometry	Exclusive left-turn lane
		Exclusive right-turn lane
		Shared lanes
Output Measures and Indicators	Measures of Effectiveness	Average delay per vehicle
	Performance Indicator	Level of service

Evaluate and Select Analysis Tool

Three analysis tools were available for the analysis. These tools are HCS (version 3.1c), CORSIM (version 4.3), and TRANSYT-7F (release 8). The capabilities of these tools were compared with the list of desired capabilities identified in Table A-9. Only those tools that provided all of the desired capabilities were considered for use in the study. Table A-10 illustrates the procedure used to assess the traffic control mode capabilities of the three analysis tools. Based on this assessment, TRANSYT-7F was eliminated from further consideration because it could not model the multi-way-stop-control alternative.

This process was repeated for the desired analysis factor capabilities. The analysis tool used for this study would need to be able to model heavy vehicle percentage, exclusive turn bays, and traffic lanes shared by through and turning vehicles. Based on an assessment of model capabilities, it was concluded that both the HCS and CORSIM tools could model all of the required analysis factors and provide the desired measure of effectiveness.

The effort required to use HCS and CORSIM was considered in making the final tool selection. In this regard, the CORSIM tool was believed to have more capability (and associated complexity) than was needed for this analysis. Also, considerable expertise had been developed by agency staff in the use of HCS for isolated intersection evaluation. Based on these considerations, the HCS was selected as the most appropriate tool for the evaluation.

Stage 2: Step 3. Conduct Evaluation

The final step in the engineering study stage was to determine the operational performance of the viable alternatives. As a first step, the need for additional information and field data was reviewed. Then, the operational performance of each alternative was quantified and its relative effectiveness was determined.

TABLE A-10 Case Study 1: Traffic control mode check sheet

Traffic Control Mode		Applies (Y/N)	Analysis Tool			Applicable Tools
			Isolated Only		Non-Isolated	
			HCS	CORSIM	T-7F	
Unsignalized	Two-way stop control	*Y*	✔	✔	✔	*HCS, CORSIM, T-7F*
	Two-way yield control	*N*	--	✔	--	
	Multi-way stop control	*Y*	✔	✔	--	*HCS, CORSIM*
	Roundabout	*N*	*v/c only*	--	--	
Signalized	Pretimed signal control	*N*	✔	✔	✔D	
	Actuated signal control	*Y*	✔	✔	✔	*HCS, CORSIM, T-7F*
	Signal coordination	*N*	--	✔	✔D	
Available Tools (tools applicable to all desired modes):						*HCS, CORSIM*

Notes:
✔ - Tool explicitly models this control mode and its effect.
-- - Tool does not explicitly model this control mode or its effect.
D - Tool can determine the optimal signal phase duration thereby eliminating the need for signal timing plan input.

Gather Information

The need for additional data was assessed at this point in the study. This assessment included consideration of the input data requirements of the chosen analysis tool (i.e., HCS) and the data needed for the benefit-versus-cost analysis. Based on this assessment, it was concluded that additional data did not need to be gathered.

Evaluate Operational Performance

Design Alternatives. Some preliminary design decisions were made regarding the operation and the geometry of the alternatives prior to their evaluation. Most of this effort was devoted to the development of a design and timing plan for the traffic signal control alternative. These design decisions are documented in this section.

Signal Alternative. The signal design process included a determination of the intersection geometry and its signal timing. Guidelines in Chapter 3 of the *ESG* indicate that an exclusive left-turn lane (or bay) may be needed when the left-turn volume exceeds 100 veh/h during the peak hour. Because the left-turn volume of 146 veh/h exceeds this threshold value, a left-turn bay on the southbound approach is included in the signal alternative.

Guidelines regarding the need for right-turn lanes are provided in Figure 3-3 of the *ESG*. A check of these guidelines indicated that a right-turn lane was not needed for the signal alternative and that one through lane on each approach would be adequate.

Guidance regarding the operation of the signal controller is provided in Table 3-5 of the *ESG*. This guidance indicated that actuated operation is appropriate for isolated intersections.

Guidance regarding the need for left-turn phasing is provided in Figure 3-5 of the *ESG*. A check of the information in this figure indicated that left-turn phasing was not needed. Thus, the signal phase sequence should consist of two through phases: one for the northbound and southbound approaches and one phase for the westbound approach.

Traffic volumes recorded for the afternoon peak period were used to determine reasonable controller settings. The analysis of the afternoon peak hour volumes is documented in Tables A-11 and A-12. The settings determined from this analysis were reasoned to be sufficiently accurate for the engineering study evaluation; however, it was recognized that they might need to be refined if the signal alternative is ultimately selected.

The HCS does not provide for the direct input of minimum and maximum green times; instead, it requires entry of average green interval duration when evaluating actuated phases. These durations were computed using the procedures described in Chapter 16, Appendix II, of the *Highway Capacity Manual 2000* (*A-3*). Once computed, they were used with the HCS to determine the average delay during each analysis period.

TWSC-1 Alternative. According to Table 2-13 of the *ESG*, the right-turn bay length on the minor road should be long enough to provide for right-turn vehicle storage. Figure 2-8 of the *ESG* indicated that right-turn bay storage length should be at least 25 ft. However, agency practice is to use 50 ft as a minimum bay length.

TWSC-2 Alternative. The length of the major-road left-turn bay at the unsignalized intersection was based on consideration of left-turn volume and approach speed. According to Table 2-13 of the *ESG*, a left-turn bay should be long enough to provide for the deceleration and storage of left-turn vehicles. Figure 2-7 indicated that a storage length of about 30 ft would be necessary. Also, Figure 2-9 indicated that an additional 290 ft was needed to provide adequate deceleration distance for an 85th percentile speed of 50 mph. Thus, the major-road left-turn bay was designed to be 320 ft in length.

TWSC-3 Alternative. Table 2-13 of the *ESG* provided some information regarding the appropriate length of a right-turn bay on the major road. According to this table, a right-turn bay should be long enough to provide for the deceleration of right-turn vehicles. As noted in the previous section, a bay length of about 290 ft is needed for turn vehicle deceleration. Thus, the major-road right-turn lane was designed to be 290 ft in length.

Evaluate Alternatives. The HCS was used to evaluate each viable alternative for the morning peak hour, afternoon peak hour, and the off-peak hour. The individual movement delays were highest during the morning peak hour; these delays are summarized in Table A-13.

The data in Table A-13 indicate that the more complex alternatives (i.e., Signal and TWSC-3) offer the least overall delay during the morning peak hour. However, the low delay for the Signal alternative is achieved at the "expense" of increased delay to the major-road through and right-turn movements. It should be noted that the delays to through vehicles due to right-turns from the major road were not estimated by the software; however, they were observed to be quite small and their omission from the delay summary was determined to have negligible effect on the evaluation.

The variation in intersection delay over time is shown in Figure A-5. The trends in this figure indicate that the multiway-stop-control alternative (MWSC) is consistently associated with the largest overall delay, reaching level of service (LOS) D during the peak hour. In contrast, the TWSC-3 alternative (i.e., add a right-turn bay on the minor road and left- and right-turn bays on the major road) would yield the lowest delay. The Signal alternative had the least delay variation throughout the day; however, it causes more delay during the off-peak hour than most of the other alternatives.

Determine Alternative Effectiveness

The findings from the operations analysis were evaluated to determine the effectiveness of each alternative. This deter-

TABLE A-11 Case Study 1: Critical volume worksheet

CRITICAL VOLUME WORKSHEET								
General Information								
Location: *CR 21 and CR 27*					**Analysis Period:** *4:00 p.m. to 5:00 p.m.*			
Volume and Lane Geometry Input								
Approach:	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Movement, No.: [1]	LT, 5	TH+RT, 2	LT, 1	TH+RT, 6	LT, 3	TH+RT, 8	LT, 7	TH+RT, 4
Volume (v_i), veh/h i = 1, 2, 3, ... 8			*140*	*65*		*291*	*143*	*464*
Lanes (n_i)			*0*	*1*		*1*	*1*	*1*
Phase Sequence	**1 Phase** (protected through & permitted left)				**1 Phase** (protected through & permitted left)			
Opposing Volume ($v_{o,i}$), veh/h	v_6 =		v_2 = *0*		v_4 =		v_8 = *291*	
LT equivalence ($E_{L,i}$) (Fig. 3-8)		1.0	*1.38*	1.0		1.0	*1.70*	1.0
Sneakers (S_i), veh/h	90	0.0	90	0.0	90	0.0	90	0.0
Adjusted volume (v^*_i) [= E_L (v_i - S_i) $\geq$ 0.0]			*69*	*65*		*291*	*90*	*464*
Lane volume ($v_{n,i}$) [= v^*_i/ n_i]{see note 2}			*0*	*134*		*291*	*90*	*464*
Critical volumes (v_c), veh/h/ln	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) = *134*				Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) = *464*			
	2 Phase (with protected-plus-permitted left)				**2 Phase** (with protected-plus-permitted left)			
Permitted capacity ($c_{p,i}$), veh/h	60	0.0	60	0.0	60	0.0	60	0.0
Adjusted volume (v^*_i) [= (v_i - $c_{p,i}$) $\geq$ 0.0]								
Lane volume ($v_{n,i}$) [= v^*_i/ n_i], veh/h/ln								
Critical volumes (v_c) {see note 3}, veh/h/ln	Larger of: ($v_{n,1}$, $v_{n,5}$)=		Larger of: ($v_{n,2}$, $v_{n,6}$)=		Larger of: ($v_{n,3}$, $v_{n,7}$)=		Larger of: ($v_{n,4}$, $v_{n,8}$)=	
	2 Phase (with protected-only left)				**2 Phase** (with protected-only left)			
Lane volume ($v_{n,i}$) [= v_i/ n_i], veh/h/ln								
Critical volumes (v_c) {see note 3}, veh/h/ln	Larger of: ($v_{n,1}$, $v_{n,5}$)=		Larger of: ($v_{n,2}$, $v_{n,6}$)=		Larger of: ($v_{n,3}$, $v_{n,7}$)=		Larger of: ($v_{n,4}$, $v_{n,8}$)=	

Notes:

1 - Numbers follow NEMA movement-based phase numbering system (LT = left-turn, TH = through, RT = right-turn).

2 - If there is no left-turn lane for a given approach, then the lane volume for the left-turn movement equals 0.0 and the lane volume for the through movement is based on the total, adjusted approach volume. For example, if the eastbound approach has no left-turn lane (i.e., n_5 = 0), then $v_{n,5}$ = 0.0 and $v_{n,2} = (v^*_5 + v^*_2)/ n_2$.

3 - Critical volume for protected left-turn phases is based on the following assumptions: (1) left-turn phases lead adjacent through phases, (2) one or more exclusive left-turn lanes exist, and (3) both left-turn movements are protected.

mination was made based on the definitions of "acceptable level of service" and "acceptable operation" as defined in Chapter 3 of the *ESG*. Table A-14 summarizes the effectiveness of each alternative.

Only two alternatives were determined to be ineffective. They are (1) the current intersection alternative (TWSC-0) and (2) the multi-way-stop-control alternative (MWSC). The signal alternative and all three of the turn-bay-related alternatives were determined to be effective.

STAGE 3: ALTERNATIVE SELECTION

During the alternative selection stage, the alternatives advanced from the engineering study stage are reviewed for

TABLE A-12 Case Study 1: Controller setting worksheet

CONTROLLER SETTING WORKSHEET								
General Information								
Location: *CR 21 and CR 27*					**Analysis Period:** *4:00 p.m. to 5:00 p.m.*			
Change Interval and Minimum Green								
Approach:	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Movement, No.: [1]	LT, 5	TH+RT,2	LT, 1	TH+RT,6	LT, 3	TH+RT,8	LT, 7	TH+RT,4
Yellow + all-red (Y_i), s (Table 3-6) {see note 2}	Y_2 =		Y_6 = ***5***	***5***	Y_8 =	***5***	Y_4 = ***5***	***5***
Ped. phase time ($P_{p,i}$), s (Table 3-6) {see note 3}	0.0		0.0	***0***	0.0	***0***	0.0	***0***
Minimum green ($G_{m,i}$), s [= larger of: ($P_{p,i}$-Y_i, 8.0)]			***8***	***8***		***8***	***8***	***8***
Critical Volume Summary {see note 4}								
1 phase E-W (n_p = 2) 1 phase N-S	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) = ***134***				Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) = ***464***			
1 phase E-W (n_p = 3) 2 phases N-S	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) =				Larger of: ($v_{n,3}$,$v_{n,7}$)=		Larger of: ($v_{n,4}$,$v_{n,8}$)=	
2 phases E-W (n_p = 3) 1 phase N-S	Larger of: ($v_{n,1}$,$v_{n,5}$)=		Larger of: ($v_{n,2}$,$v_{n,6}$)=		Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) =			
2 phases E-W (n_p = 4) 2 phases N-S	Larger of: ($v_{n,1}$,$v_{n,5}$)=		Larger of: ($v_{n,2}$,$v_{n,6}$)=		Larger of: ($v_{n,3}$,$v_{n,7}$)=		Larger of: ($v_{n,4}$,$v_{n,8}$)=	
Sum of critical volumes (Σv_c), veh/h/ln	Σv_c = ***598*** No. phases (n_p) = ***2***				Min. Delay Cycle (C_o) = ***30*** (Fig. 3-6) Cycle Length (C) = ***30***			
Pretimed Phases	Pretimed (or non-actuated) phase time				Pretimed (or non-actuated) phase time			
Critical volumes by phase ($v_{c,i}$), veh/h/ln {see note 5}								
Green duration (G), s [= $v_{c,i}/\Sigma v_c$ (C - 4n_p) + 4 -Y_i]								
Actuated Phases	Actuated phase maximum green setting				Actuated phase maximum green setting			
Lane volume ($v_{n,i}$), veh/h/ln (from Critical Vol. Wksht.)				***134***		***291***		***464***
Min.-delay green ($G_{o,i}$) s [= $v_{n,i}$/ Σv_c (C_o - 4n_p) + 4 -Y_i]				***3.9***		***9.7***		***16.1***
Maximum green setting, s [= larger of:($G_{m,i}$+12, 1.3$G_{o,i}$)]				***20***		***20***		***21***
Unit extension, s (Table 3-10)				***2.1***		***2.1***		***2.1***

Notes:

1 - Numbers follow NEMA movement-based phase numbering system (LT = left-turn, TH = through, RT = right-turn). Complete the columns for the left-turn movements (i = 1, 3, 5, 7) *only* if a corresponding left-turn phase exists.

2 - Compute the change interval ($Y + AR$) for the through phases only. If a left-turn phase exists then set its change interval equal to that associated with the adjacent through movement.

3 - If pedestrians are served on a through phase that does not have pedestrian detection (i.e., no ped. button or ped. signal) then use Table 3-6 to determine the minimum pedestrian phase time; otherwise, use P_p = 0.0 s.

4 - Obtain from the Critical Volume Worksheet the critical volume that is associated with each movement. Only one phase combination (or row) should be used.

5 - Record the critical phase volume in all cells that correspond to the movements served.

their potential impact on traffic operations, safety, and the environment. The alternative selection stage consists of the following three steps:

1. Identify impacts,
2. Select best alternative, and
3. Document study.

Stage 3: Step 1. Identify Impacts

Identify Decision Factors

Several factors were identified that might influence alternative selection. These decision factors include traffic operations, traffic safety, and direct cost. Impacts to the envi-

TABLE A-13 Case Study 1: Delay summary for peak traffic hour

Alternative	Movement Delay (s/veh)					Intersection Delay (s/veh)
	Northbound	Southbound		Westbound		
	Through + Right-turn	Left-turn	Through	Left-turn	Right-turn	
1. TWSC-0	0	8	2	98		22
2. MWSC	11	36		12		26
3. Signal	9	12	10	12		11
4. TWSC-1	0	8	2	89	9	16
5. TWSC-2	0	8	0	75	9	13
6. TWSC-3	0	8	0	58	9	11

ronment and area aesthetics are negligible for the proposed alternatives.

Assess Degree of Impact

Each alternative's impact on traffic operations, safety, and cost was evaluated. The traffic operations impacts were quantified using motorist delay. The total delay during the average day was computed by assuming that each peak-hour delay was incurred for 2 hours and that the off-peak delay was incurred for 18 hours. For the Signal alternative, total delays of 3.1, 1.4, and 3.1 veh-h/h were predicted for the morning peak hour, off-peak hour, and afternoon peak hour, respectively. The average-day delay for the Signal alternative was estimated as 38 veh-h/day ($= 2 * 3.1 + 18 * 1.4 + 2 * 3.1$). The total delay for the other alternatives was computed in a similar manner.

The annual number of crashes was estimated using average crash rates for intersections with characteristics similar to those of the subject intersection. This analysis suggested that the Signal alternative would have the most crashes at 9 per year. The two-way-stop-control alternatives were estimated to have lower crash frequencies at 4 or 5 crashes per year, depending on whether turn bays are present on the major road. The initial and annual costs of each alternative

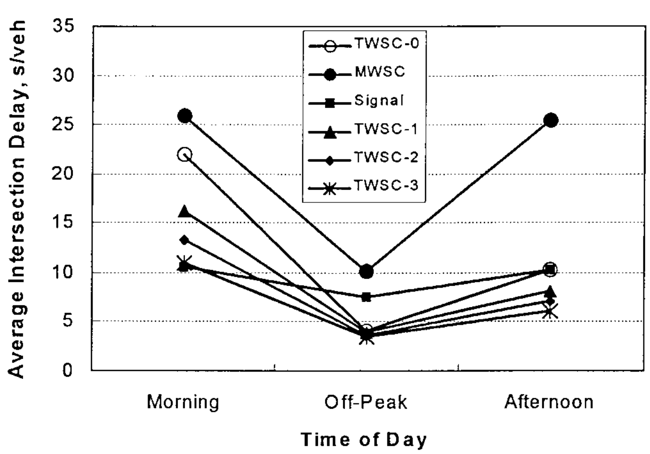


Figure A-5. Case Study 1: Average intersection delay.

were estimated using costs developed for similar intersections during the past year. The estimated impacts of the alternatives are summarized in Table A-15.

Stage 3: Step 2. Select Best Alternative

As a first activity of this step, the factor weights were identified. These weights would be used to aggregate the impacts of the various factors identified in Step 1. The weights selected are listed in Column 4 of Table A-15. The weight used for the delay impact is based on an estimate of the annual cost of time per vehicle (in thousands of dollars). It assumes that there are 300 equivalent average days per year and that the cost of time is $15 per vehicle-hour (i.e., $4.5 = 300 * 15 / 1000$). Thus, the product of this weight and total delay yields an estimate of the annual cost of delay per year for a given alternative.

The weight used for crash frequency reflects an equivalent cost for the average crash (in thousands of dollars) and reflects the distribution of fatal, injury, and property damage crashes at intersections. The initial costs (in thousands of dollars) were given a weight of only 0.1 to reflect their annualization over a 15-year period. These weights were used to compute the weighted total shown in the last row of the table. This total represents the annual cost of each alternative (in thousands of dollars).

The total impact values listed in the last row of Table A-15 indicate that the TWSC-3 alternative (i.e., add a right-turn bay on the minor road and left- and right-turn bays on the major road) has the lowest overall impact. In contrast, the Signal alternative has the largest impact due in part to its large initial cost, its large number of expected crashes, and its large total delay. Based on this analysis, the TWSC-3 alternative was selected as the best alternative. A sketch of the recommended alternative is shown in Figure A-6.

Stage 3: Step 3. Document Study

A report documenting the results of the engineering study was prepared and submitted.

TABLE A-14 Case Study 1: Alternative effectiveness

Alternative	Reduces Average Intersection Delay?	Acceptable Intersection Level of Service?	Acceptable Movement Operation?	Effective Alternative?
1. TWSC-0	n.a.	Yes	No	No
2. MWSC	No	Yes	Yes	No
3. Signal	Yes	Yes	Yes	Yes
4. TWSC-1	Yes	Yes	Yes	Yes
5. TWSC-2	Yes	Yes	Yes	Yes
6. TWSC-3	Yes	Yes	Yes	Yes

Note:
n.a. - not applicable

TABLE A-15 Case Study 1: Alternative impacts

Factor	Description	Units	Weight	Alternative Impact			
				Signal	TWSC-1	TWSC-2	TWSC-3
Traffic Operations	Motorist delay	veh-h/day	4.5	38	28	24	22
Traffic Safety	Crash frequency	crashes/year	40	9	5	4	4
Direct Cost	Initial costs	$ /1000	0.1	60	15	30	45
	Annual costs	$ /1000	1.0	2	0	0	0
			Total:	539	328	271	264

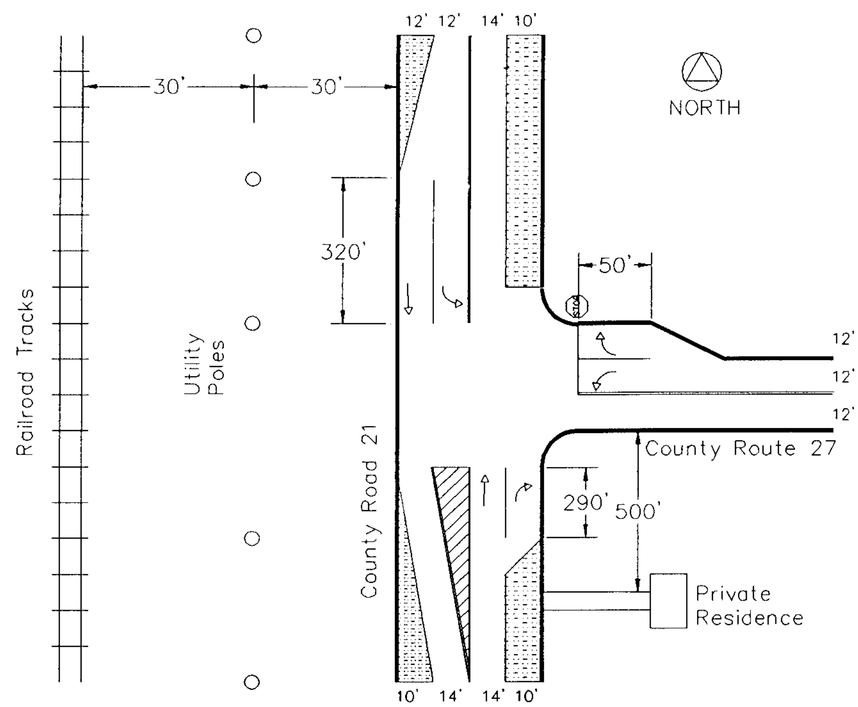


Figure A-6. Case Study 1: Recommended alternative (not to scale).

CASE STUDY 2

SYNOPSIS

This case study illustrates the application of the *Engineering Study Guide* (*ESG*) to an isolated, two-way stop-controlled intersection where minor-road drivers are experiencing excessive delay. The SIDRA software package was used to evaluate the operation of the intersection and its proposed alternatives. Conversion to traffic signal control was recommended as the best method of reducing motorist delay.

BACKGROUND

Recently, several housing developments have been built on Barracks Road near its intersection with U.S. 29. These developments have increased traffic volumes in general and truck volumes in particular (many of which are construction related). A local elected official requested the installation of a traffic signal at the intersection to reduce delay and improve safety. Based on this request, an engineering study was undertaken to determine whether improvements to the intersection were needed.

The intersection of Barracks Road and U.S. 29 is located in a rural area with a population of less than 9,000 people. The intersection has four approach legs; two opposing legs are stop-controlled. Barracks Road is the minor road; it is oriented in an east-west direction and has stop control at the intersection. U.S. 29 is the major road; it is oriented in a north-south direction. The speed limit is posted at 55 mph (90 km/h) on U.S. 29 and 30 mph (50 km/h) on Barracks Road. The geometric layout of the intersection is shown in Figure A-7.

STAGE 1: ALTERNATIVE IDENTIFICATION AND SCREENING

The first stage of the engineering assessment process involved diagnosing the problems at the intersection and determining potential corrective measures. The alternative identification and screening stage consists of the following steps:

1. Define problem and cause,
2. Select candidate alternatives, and
3. Select viable alternatives.

Stage 1: Step 1. Define Problem and Cause

Gather Information

The first step was to gather information about the intersection. This involved checking agency records and visiting the site. The data gathered during this step are summarized in the remainder of this section.

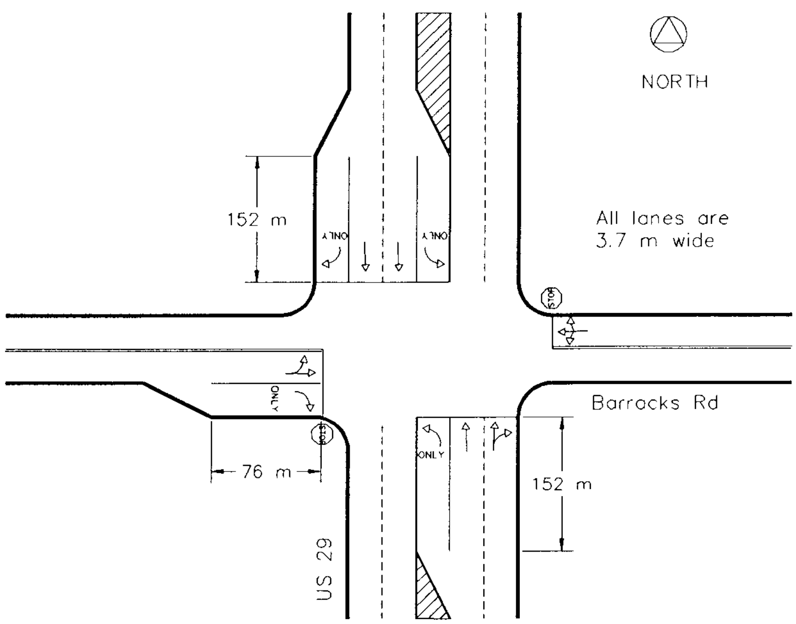


Figure A-7. Case Study 2: Existing intersection geometry (not to scale).

TABLE A-16 Case Study 2: Potential problems and causes

Potential Problem	Possible Cause
Excessive delay to the minor-road movements.	• Inadequate capacity for minor-road movements. • Inadequate separation of minor-road movements.
Some delay and conflict to major-road through and right-turn movements.	• Inadequate separation of major-road movements.

Historic Data. This intersection had not been the subject of a prior engineering study, so archival traffic data were not available. Crash data were obtained from the Accident Records Division in the state's Department of Transportation. A review of these data indicated that six crashes had occurred at the intersection in the last year (three susceptible to correction by a signal). The annual average daily traffic (AADT) is 12,000 vehicles per day on U.S. 29 and 7,000 vehicles per day on Barracks Road.

Observational Study. The intersection was visited during a typical weekday to determine whether the reported delay problems were present at the intersection. During this visit, vehicles on the minor road were observed to experience excessive delay during the peak hours. Also, some delay and conflict was observed between the northbound through and right-turn movements. An On Site Observational Report was completed to document the nature of the operational and safety problems at the intersection.

Site Survey. A site survey was also conducted during the site visit. Key geometric features of the intersection noted during this survey are summarized in Figure A-7.

Define Problem and Cause

Define Problem and Identify Cause. Based on the results of the observational study, it was found that sufficient evidence existed to justify continuing with the engineering study. As a next step in this study, Table 2-1 of the *ESG* was used to identify possible causes for the observed delays. The findings from this assessment are summarized in Table A-16.

Define Influence Area. The subject intersection's influence area was defined to consist of the intersection conflict area and a 100-m length of each intersection approach.

Stage 1: Step 2. Select Candidate Alternatives

Identify Potential Alternatives

Table 2-2 in the *ESG* was consulted to identify alternatives that could address the observed problems at the intersection. The alternatives identified in Table 2-2 that are applicable to the subject intersection are summarized in Table A-17.

Organize and Select Alternatives

The potential alternatives were subjected to a preliminary screening to eliminate any alternatives that would not be feasible at the site. Based on this screening, several alternatives were eliminated. The alternatives eliminated are listed in Table A-18.

Based on this analysis, the following alternatives were found to merit further study:

- Convert to traffic signal control,
- Add right-turn bay to major road,
- Add second lane on minor road, and
- Convert to roundabout.

Stage 1: Step 3. Select Viable Alternatives

The candidate alternatives selected in Step 2 were more closely examined in this step. This examination focused on

TABLE A-17 Case Study 2: Potential alternatives

Possible Cause	Corrective Strategy	Potential Alternatives
Inadequate capacity for minor-road movements.	Increase approach capacity.	**1. Convert to roundabout.** 2. Convert to yield control. **3. Convert to traffic signal control.** 4. Convert to multi-way stop control
Inadequate separation of minor-road movements.	Separate conflicting flows.	**1. Add a second lane on minor road.** 2. Increase right-turn radius.
Inadequate separation of major-road movements.	Separate conflicting flows.	**1. Add right-turn bay.** 2. Increase right-turn radius.

TABLE A-18 Case Study 2: Alternatives eliminated

Alternative	Reason for Elimination
Convert to yield control.	Yield control is not suitable for intersections with a major arterial.
Convert to multi-way stop control.	Major/minor volume differential too great for multi-way stop control.
Increase right-turn radius (on minor road).	Westbound right-turn currently has a large 15-m radius.
Increase right-turn radius (on major road).	Northbound right-turn currently has a large 15-m radius.

evaluating each alternative to assess its "viability" (i.e., its ability to address the observed problems effectively).

Gather Information

Identify Data. Table 2-5 in the *ESG* was used to identify the data needed to evaluate the candidate alternatives. Based on the information in this table, it was determined that the following data were needed for guideline evaluation:

- Major- and minor-road turn movement volume (8 hours),
- Heavy vehicle volume (8 hours),
- Pedestrian volume (4 hours),
- Gap frequency (4 hours),
- 85th percentile approach speed,
- Intersection sight distance,
- ✓ Area population,
- ✓ Number of lanes, and
- ✓ Crash history by type.

Of these data, information about the last three items had already been gathered in a previous step. The minimum number of hours for which data would be collected is indicated in the list above.

Collect Data. At the onset of this task, it was noted that vehicular volumes and approach speed were common to most of the alternatives. It was also noted that the satisfaction of one signal warrant would confirm the viability of the signal alternative. From this assessment, it was decided initially to collect only volume and speed data. These data would allow a check of the *Manual on Uniform Traffic Control Devices-Millennium Edition* (i.e., *MUTCD 2000*) (*A-2*) Signal Warrants 1 and 2. If these warrants were not satisfied, then additional data would be collected to allow a check of Warrants 4 and 8.

Warrant 3 was not evaluated because the intersection was not believed to represent an "unusual case." Specifically, the intersection was not near an office complex, manufacturing plant, or other facility that generated high volumes over a short time period.

Because it was unclear when the 8 peak traffic hours occurred, turn movement volumes were recorded for a total of 12 hours. The counts were obtained on a Wednesday in April; they were then adjusted to eliminate bias due to weekly and monthly variations. The estimated average-day volumes are summarized in Table A-19. A review of the hourly totals indicated that the morning peak hour occurred from 7 to 8 a.m., the representative off-peak period was reasoned to occur from 10 to 11 a.m., and the afternoon peak hour occurred from 4 to 5 p.m.

No pedestrians were observed during the study period. Heavy vehicles comprised about 30 percent of the traffic stream on the minor road and 2 percent on the major road. A spot speed study was conducted for U.S. 29; the 85th percentile speed was found to be 91 km/h.

Assess and Select Alternatives

Convert to Traffic Signal Control. The traffic signal warrants in the *MUTCD 2000* (*A-2*) were evaluated to assess the viability of the traffic signal alternative. Based on this evaluation, the following warrants were satisfied at this intersection:

- Warrant 1: Eight-hour vehicular volume (Condition A only) and
- Warrant 2: Four-hour volume.

For the warrant evaluation, the "70%" values were used because the major-road speed exceeded 70 km/hr. Condition A of Warrant 1 was satisfied for all 12 hours for which data were collected. Warrant 2 was satisfied for 11 of the 12 hours. It should be noted that the right-turn volume on the eastbound approach was not included in the approach total because right-turn vehicles were (1) provided a turn bay and (2) observed to enter the major road without conflict.

The presence of conditions that could misdirect the signal warrant check was also assessed. A review of Table 2-15 in the *ESG* revealed that two conditions exist that could affect the warrant analysis conclusions. These conditions are

- Heavy vehicles on the minor road and
- Left-turn bays on the major road.

The existence of problematic conditions required additional analysis to determine whether the conclusions from the warrant check were valid. The exclusion of the right-turn

TABLE A-19 Case Study 2: Turn movement volumes

Hour	Volume (veh)[1]													
	Northbound			Southbound			Major Total	Eastbound			Westbound			High Minor
	L	T	R	L	T	R		L	T	R	L	T	R	
6-7 a.m.	10	404	45	58	21	22	560	275	16	38	40	46	208	294
7-8 a.m.	**38**	**380**	**71**	**202**	**94**	**50**	**835**	**162**	**80**	**176**	**143**	**115**	**126**	**384**
8-9 a.m.	32	163	60	141	110	43	549	61	44	113	120	90	140	350
9-10 a.m.	57	94	41	97	86	73	448	33	39	38	36	73	95	204
10-11 a.m.	**51**	**57**	**65**	**128**	**92**	**92**	**485**	**44**	**62**	**60**	**43**	**55**	**63**	**161**
11-12 p.m.	38	86	83	163	74	76	520	37	40	82	98	77	147	322
12 a.m.-1 p.m.	79	121	85	156	79	70	590	59	41	74	80	111	96	287
1-2 p.m.	64	116	82	160	62	44	528	86	43	77	78	72	94	244
2-3 p.m.	89	148	90	177	105	85	694	42	59	73	94	86	72	252
3-4 p.m.	131	155	74	234	154	118	866	91	67	81	72	85	87	244
4-5 p.m.	**33**	**297**	**32**	**188**	**642**	**31**	**1,223**	**147**	**51**	**27**	**90**	**48**	**100**	**238**
5-6 p.m.	123	163	144	283	158	191	1,062	120	43	79	134	60	101	295

Note:
1 - Movement types: L = left-turn, T = through, R = right-turn.

volume on the eastbound approach was reasoned to be consistent with *MUTCD 2000* guidance (*A-2, p. 4-C2*) and, thus, it was not believed to represent a problematic condition.

Frequent heavy vehicles on the minor road were a point of concern because their representation is much larger than typically found at most intersections. In fact, guidance provided in Chapter 2 of the *ESG* indicated that intersections with more than 5 percent heavy vehicles are atypical and may not be fully reflected in the warrants. Because the subject intersection has 30 percent heavy vehicles, it was decided that the operational effects of heavy vehicles would be explicitly considered in the evaluation of the signal alternative.

Chapter 2 of the *ESG* indicated that a left-turn bay on the major road may affect the warrant check when the approach has only one through traffic lane. In this instance, the approach had two through lanes, so the effect of the left-turn bay was determined to be insignificant, as it relates to the accuracy of the warrant check conclusion.

In summary, the warrant analysis indicated that a traffic signal may improve traffic operations at the intersection. However, the guideline addressing the effect of "heavy vehicle percentage" indicated that the large number of trucks at the intersection may have an impact on the accuracy of the warrant check conclusion. Therefore, the operation of the traffic signal alternative would be examined in greater detail in the engineering study stage. Moreover, the examination would explicitly include the effect of trucks.

Add Right-Turn Bay to Major Road. Figure 2-6 in the *ESG* was examined to determine whether an exclusive right-turn bay would be beneficial on the northbound intersection approach. The examination was based on a major-road speed of 90 km/h (55 mph) and the afternoon peak hour volumes. From this examination, it was concluded that the right-turn bay would be beneficial.

Add Second Lane on Minor Road. Figure 2-4 in the *ESG* was examined to determine whether adding an exclusive turn bay to the westbound approach would improve traffic operations at the intersection. The examination was based on the afternoon peak hour volumes and a westbound right-turn volume that constituted 42 percent of the approach volume. Based on this evaluation, it was concluded that an additional lane on the westbound approach would be beneficial. Site conditions indicated that this lane could most easily be added as a right-turn bay.

Convert to Roundabout. Table 2-12 in the *ESG* was evaluated to determine whether a roundabout was a viable alternative for this intersection. The results of this evaluation are shown in Table A-20.

The response to Question 2 in Table A-20 required an estimate of the daily entering volume and the roundabout's maximum service volume. Daily entering volume was estimated as 19,000 veh/d based on AADTs of 12,000 and 7,000 veh/d for the major and minor roads, respectively. Maximum service volume was estimated as 41,500 veh/d using the equation in the footnote to Table A-20. This estimate was based on 2-lane roundabout approaches, 63 percent of the volume entering on the major road, and 34 percent left-turns at the intersection. The ratio of major-road-to-minor-road volume was computed as 1.7 (= 12/7).

As indicated by the last column of Table A-20, the roundabout was found to be a viable alternative for all of the fac-

TABLE A-20 Case Study 2: Roundabout evaluation worksheet

Question	Y/N
1. Will operation as uncontrolled or two-way stop-controlled intersection yield unacceptable delay?	*Y*
2. Is the daily entering volume less than the maximum daily service volume for a roundabout?[1]	*Y*
3. Is the subject junction located outside of a coordinated signal network?	*Y*
4. Is the ratio of major-road-to-minor-road volume less than 5.0?	*Y*
5. Is the entering driver's view free of sight obstructions (e.g., due to grade, curvature, vegetation, etc.)?	*Y*
6. Will the subject junction be infrequently used by large or over-sized trucks?	*N*
7. Will the subject junction be infrequently used by pedestrians and bicyclists?	*Y*

Note:

1 - Maximum service volume (4-legs), veh/d $= 3600 + 9000\,lanes\left(1 + \frac{81}{major}\right) - 94\,lefts$, where *lanes* = number of roundabout entry lanes per approach, *major* = percent of volume entering on the major-road approaches, and *lefts* = percent of left-turns at junction. Maximum service volume (3-legs) = 0.75 * maximum service volume (4-legs).

tors considered except "heavy vehicle frequency" (i.e., Question 6). Thus, there was some concern about the ability of the roundabout design to serve frequent truck traffic. There was also some concern about the suitability of the roundabout to a rural, high-speed location. Despite these concerns, the roundabout alternative was advanced to the engineering study stage.

Summary of Viable Alternatives

Based on the assessment of viable alternatives, all four candidate alternatives identified in Step 2 were selected for more detailed analysis in the next stage of the assessment process. These alternatives are summarized below:

- **TWSC**: Base case (existing intersection with no improvements);
- **Signal:** Conversion to traffic signal control;
- **Bay:** Two-way stop control with right-turn bays on westbound and northbound approaches; and
- **Roundabout:** Conversion to a two-lane roundabout.

The major- and minor-road right-turn bay alternatives were combined into one alternative. This alternative is denoted in the list above as "Bay." The designation shown in bold type is used in subsequent sections to refer to the various alternatives.

STAGE 2: ENGINEERING STUDY

The operational performance of the alternatives identified in the alternative identification and screening stage were evaluated in the engineering study stage. This stage consists of three steps:

1. Determine study type,
2. Select analysis tool, and
3. Conduct evaluation.

Stage 2: Step 1. Determine Study Type

The first step of the engineering study stage required a determination of the type of study needed. Initially, the relationship between the subject intersection and any upstream or downstream signalized intersection was evaluated. Then, the type of evaluation needed was determined.

Determine Type of Operation

The interaction between the subject intersection and any adjacent signalized intersection was evaluated to determine whether the operation of the adjacent intersection should be considered in the analysis. An investigation of the location of these intersections revealed that there are no intersections within several kilometers of the subject intersection. Figure 3-1 of the *ESG* indicates that distances exceeding 730 m (for a major-road peak hour volume of 1,223 veh/h) are sufficient to isolate the subject intersection from the adjacent signalized intersections.

Determine Type of Evaluation

Next, the intersection was examined to determine whether a formal evaluation of alternatives was required. The *ESG* suggests that an informal evaluation (i.e., implementation and field study) is possible when there is only one viable alternative applicable at an isolated intersection. However, the fact that the subject intersection is associated with several viable alternatives required the conduct of a formal evaluation.

Stage 2: Step 2. Select Analysis Tool

During this step, an analysis tool was selected for evaluating the operation of the subject intersection and its alternatives. This selection was based on an identification of the desired analysis tool capabilities and a comparison of this list with the actual capabilities of the available tools. The tool selected for use was the one that provided all of the desired capabilities.

Identify Desired Capabilities

Table 3-1 in the *ESG* was examined to determine which analysis tool capabilities would be required for this evaluation. Based on this examination, it was determined that the analysis tool selected must be able to evaluate the traffic control modes used with the alternatives (i.e., two-way stop control, actuated traffic signal control, and roundabout operation). It was also determined that the analysis tool must be able to evaluate the percentage of heavy vehicles, left- and right-turn bays, and shared-lane approaches. Finally, it was decided that the analysis tool should provide a "level of service" indication.

Evaluate and Select Analysis Tool

Three analysis tools were available for the evaluation: HCS (version 3.1c), CORSIM (version 4.3), and SIDRA (version 5.02a). Each analysis tool was evaluated for its ability to provide the desired capabilities. CORSIM was eliminated because it cannot be used to evaluate roundabout operations. The HCS was also eliminated because it can only be used to estimate roundabout capacity. SIDRA was determined to be the only analysis tool that could be used to evaluate all of the alternatives.

Stage 2: Step 3. Conduct Evaluation

The final step in the engineering study stage was to determine the operational performance of the viable alternatives. Before conducting this evaluation, the need for additional information and field data was reviewed. Then, the operational performance of each alternative was quantified and its relative effectiveness was determined.

Gather Information

The need for additional data was assessed at this point in the study. This assessment focused on the input data requirements of the chosen analysis tool (i.e., SIDRA). The initial data collection effort was determined to be thorough enough that all necessary data were available. Based on this finding, it was concluded that additional data did not need to be gathered.

Evaluate Operational Performance

Design Alternatives. Some preliminary design decisions were made regarding the operation and the geometry of the alternatives before they were evaluated. Most of this effort was devoted to the development of a design and timing plan for the traffic signal control alternative. However, some effort was also expended on the design of the roundabout.

Signal Alternative. The signal design process included a determination of the intersection geometry and its signal timing. Guidelines in Chapter 3 of the *ESG* indicated that an exclusive left-turn lane (or bay) may be needed when the left-turn volume exceeds 100 veh/h. However, it also notes that this criterion may not be appropriate when the opposing and adjacent through volumes are less than 450 veh/h. The east and west approaches of the intersection satisfy this criterion for the afternoon and morning peak hours, respectively. However, the through volumes were exceptionally light during these hours. Therefore, it was decided that left-turn bays on the eastbound and westbound approaches would not be included in the intersection design for the signal alternative.

Guidelines regarding the need for right-turn lanes are provided in Figure 3-3 of the *ESG*. A check of these guidelines indicated that right-turn lanes were not needed for the signal alternative.

Guidance regarding the operation of the signal controller is provided in Table 3-5 of the *ESG*. This guidance indicated that actuated operation is appropriate for isolated intersections.

Guidance regarding the need for left-turn phasing is provided in Figure 3-5 of the *ESG*. A check of the information in this figure indicated that left-turn phasing was not needed. Thus, the signal phase sequence should consist of two through phases: one for the north and southbound approaches and one phase for the east and westbound approaches.

Traffic volumes recorded for the afternoon peak period were used to determine the controller settings. This analysis followed the guidelines associated with Tables 3-7 and 3-8 in the *ESG*. The settings determined from this analysis were reasoned to be sufficiently accurate for the engineering study evaluation; however, it was recognized that they may need to be refined if the signal alternative is selected.

Bay Alternative - Major-Road Approach. Table 2-13 of the *ESG* provided some information regarding the appropriate length of a right-turn bay on the major road. According to this table, a right-turn bay should be long enough to provide for the deceleration of right-turn vehicles. Figure 2-9 indicated that about 115 m was needed to provide adequate deceleration distance when the 85th percentile speed is 91 km/h. Thus, the northbound major-road right-turn bay was designed to be 115 m in length.

Bay Alternative - Minor-Road Approach. According to Table 2-13 of the *ESG*, the right-turn bay length on the westbound approach should be long enough to provide for right-turn vehicle storage. Figure 2-8 of the *ESG* was consulted for each of the three analysis hours. The morning peak hour yielded the greatest storage length of 15 m. Thus, the westbound minor road right-turn bay was designed to be 15 m in length.

TABLE A-21 Case Study 2: Delay summary for peak traffic hour

Alternative	Movement Delay (s/veh)[1]												Intersection Delay (s/veh)
	Northbound			Southbound			Eastbound			Westbound			
	L	T	R	L	T	R	L	T	R	L	T	R	
1. TWSC	5	0	0	2	0	0	630		8	249			110
2. Signal	9	5	4	9	8	1	7		2	4			7
3. Bay	5	0	0	2	0	0	216		7	607		8	90
4. Roundabout	2	2	2	1	1	1	4	4	4	2	2	2	2

Note:
1 - Movement types: L = left-turn; T = through; R = right-turn.

Roundabout Alternative. It was determined that the roundabout could be designed to accommodate the large number of heavy vehicles. The design guidelines provided in *Roundabouts: An Informational Guide* (*A-4*) were used for this purpose. This document indicated that a two-lane roundabout with an inscribed circle diameter of 60 m (and central island diameter of 50 m) could accommodate the travel paths of turning trucks.

Evaluate Alternatives. SIDRA was used to evaluate each alternative for the morning peak, off-peak, and afternoon peak hours. The individual movement delays predicted for the afternoon peak traffic hour (i.e., 4:00 to 5:00 p.m.) are summarized in Table A-21. It should be noted that similar levels of delay were observed during the morning peak hour.

The data in Table A-21 indicate that the two more restrictive alternatives (i.e., Signal and Roundabout) offer the least overall delay. This low delay would be achieved at the "expense" of increased delay to the major-road through and right-turn movements. However, this increase was very small when compared with the decrease in delay to minor-road movements.

The variation in intersection delay over time is shown in Figure A-8. The trends in this figure indicate that the existing intersection (TWSC) resulted in the largest delay. In contrast, the roundabout resulted in the smallest delay.

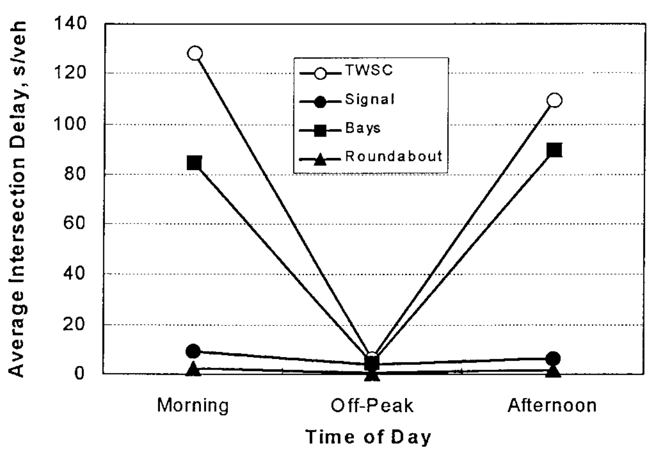


Figure A-8. Case Study 2: Average intersection delay.

Determine Alternative Effectiveness

The findings from the operations analysis were evaluated to determine whether each alternative would serve traffic effectively. This determination was made based on the definitions of "acceptable level of service" and "acceptable operation" as defined in Chapter 3 of the *ESG*. Table A-22 summarizes the results of this evaluation.

Only the Signal and Roundabout alternatives were found to satisfy all three of the "effectiveness" criteria. The existing intersection and the turn bay alternative did not yield minor-road delays that were low enough to be considered acceptable.

STAGE 3: ALTERNATIVE SELECTION

During the alternative selection stage, the alternatives advanced from the engineering study stage are reviewed for their potential impact on traffic operations, safety, and the environment. The alternative selection stage consists of the following three steps:

1. Identify impacts,
2. Select best alternative, and
3. Document study.

Stage 3: Step 1. Identify Impacts

Identify Decision Factors

Several factors were identified that might influence alternative selection. These decision factors included traffic operations, traffic safety, and direct cost. Impacts to the environment and area aesthetics were considered but were largely unaffected by the proposed alternatives.

Assess Degree of Impact

Each alternative's impact on traffic operations, safety, and cost was evaluated. The traffic operations impacts were quantified using total delay during the average day. An estimate of annual crash frequency was used to quantify safety

TABLE A-22 Case Study 2: Alternative effectiveness

Alternative	Reduces Average Intersection Delay?	Acceptable Level of Service?	Acceptable Operation?	Effective Alternative?
1. TWSC	n.a.	No	No	No
2. Signal	Yes	Yes	Yes	Yes
3. Bay	Yes	No	No	No
4. Roundabout	Yes	Yes	Yes	Yes

Note:
n.a. - not applicable

impacts. Estimates of the initial construction and annual maintenance costs were used to estimate the direct-cost impacts. The total delay for the average day was estimated from the evaluation conducted in the engineering study stage. It was assumed that the morning peak hour, off-peak hours, and afternoon peak hour existed for 2, 18, and 2 hours, respectively, during the average day.

The crash frequency for the roundabout was difficult to estimate as there was little data available for reference. A recent report by Robinson et al. (*A-4*) indicated that multilane roundabouts have more conflicts than single-lane roundabouts and cite data indicating that crash frequency reduction may be small for this location. Given the high speed on the major-road approach, it was conservatively assumed that there would be no change in annual crash frequency with the roundabout. On the other hand, data from similar signalized intersections indicated that it was likely that crash frequency would be reduced (to about four crashes per year) at this intersection, if it were signalized. Finally, the cost of constructing the roundabout was estimated to be $400,000 due to the large amount of right-of-way, design complexity, and new pavement area needed. Table A-23 illustrates the findings from the impact assessment.

Stage 3: Step 2. Select Best Alternative

The best overall alternative was selected by choosing impact weights that reflect an economic worth for delay and crashes. These weights were established by the agency for previous projects and found to yield reasonable results. The weights selected for this study are listed in Column 4 of Table A-23.

Weighted impact totals for each alternative are shown in the last row of Table A-23. These totals indicate that the signal alternative is slightly more desirable than the multilane roundabout alternative. The roundabout is likely to perform better operationally. However, there was some concern about the ability of the roundabout to improve safety at this intersection. The anticipated cost to construct the roundabout is also expected to be much larger than the signal due to the roundabout's larger size and geometric design complexities.

Based on this analysis, the Signal alternative was selected as the best overall alternative for this intersection.

Stage 3: Step 3. Document Study

A report documenting the results of the engineering study was prepared and submitted.

TABLE A-23 Case Study 2: Alternative impacts

Factor	Description	Units	Weight	Impact	
				Roundabout	Signal
Traffic Operations	Motorist delay	veh-h/day	4.5	7	29
Traffic Safety	Crash frequency	crashes/year	40	6	4
Direct Cost	Initial costs	$ /1000	0.1	400	60
	Annual costs	$ /1000	1.0	2	2
			Total:	314	299

CASE STUDY 3

SYNOPSIS

This case study illustrates the application of the *Engineering Study Guide* (*ESG*) to an isolated, stop-controlled intersection where minor-street drivers are experiencing excessive delay. These delays are partially created by frequent on-street parking maneuvers occurring on the minor street. CORSIM was used to evaluate the intersection. The removal of some on-street parking on the minor street and its replacement with a right-turn bay was recommended as the best method for alleviating motorist delay.

BACKGROUND

The intersection of Market Street and Maple Avenue has been the focus of ongoing complaint about excessive delay during peak hours and unsafe parking maneuvers. Recent increases in traffic demand (due to growth in the downtown area) have amplified these problems and the corresponding number of complaints. The number of complaints received in recent weeks has prompted the need for an engineering evaluation of traffic conditions at the intersection.

The intersection is on the edge of the downtown area in a city of 40,000 people. The intersection has three approach legs with stop control on Maple Avenue. Market Street is the major street; it is oriented in an east-west direction. Maple Avenue is the minor street; it is oriented in the north-south direction. Both streets have a posted speed limit of 35 mph. The westbound approach has an exclusive left-turn bay; all approaches have one through lane and no right-turn bays. The layout of the intersection is shown in Figure A-9.

STAGE 1: ALTERNATIVE IDENTIFICATION AND SCREENING

The first stage of the engineering assessment process involved diagnosing the problem at the intersection and determining potential corrective measures. Candidate alternatives were identified and given a preliminary screening to determine whether they represented viable solutions to the problem. The alternative identification and screening stage consists of the following steps:

1. Define problem and cause,
2. Select candidate alternatives, and
3. Select viable alternatives.

Stage 1: Step 1. Define Problem and Cause

Gather Information

The first step was to gather information about the intersection. This involved checking agency records and visiting the site. The data gathered during this step are summarized in the remainder of this section.

Historic Data. The intersection had not been the subject of a prior engineering study, so archival traffic data were not available. Crash data were obtained from the Accident Records Division of the Department of Public Works. These data indicated that a total of four collisions occurred at the intersection in the last year (two susceptible to correction by a signal). Regional traffic counts recorded 1 year earlier indicated an annual average daily traffic (AADT) of 12,600 vehi-

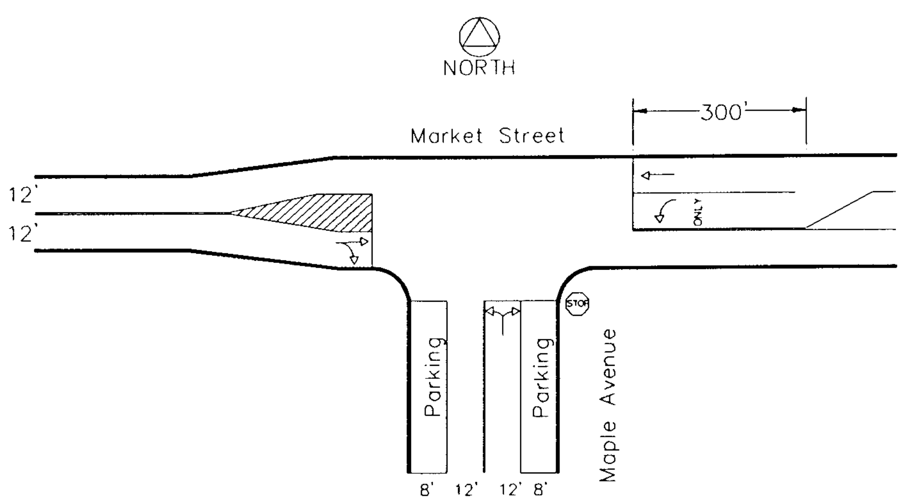


Figure A-9. Case Study 3: Existing intersection geometry (not to scale).

TABLE A-24 Case Study 3: Potential problems and causes

Potential Problem	Possible Cause
Excessive delay and conflict on the minor road.	• Excessive on-street parking activity on minor road.
Excessive delay to minor-road movements.	• Inadequate separation of minor-road movements.
Excessive delay to left-turn movements.	• Inadequate capacity for left-turn movements.

cles per day on Market Street and 11,500 vehicles per day on Maple Avenue.

Observational Study. The intersection was visited during a typical weekday to determine whether the problems reported were present at the intersection. The observational study revealed that the vehicles on the minor street experienced excessive delay during the peak hours. Most of the delay was due to the difficulty that northbound left-turn drivers had finding an adequate gap in traffic on Market Street. This difficulty translated into significant delay to both left-turn and right-turn drivers because they shared the same approach lane. Some conflict and delay was also observed to result from frequent parallel parking maneuvers on the minor-street approach.

Site Survey. A site survey was also conducted during the site visit. Key geometric features of the intersection noted during this survey are summarized in Figure A-9. Also noted during this visit were the characteristics of the on-street parking on Maple Avenue. Parallel parking is permitted along both sides of Maple Avenue. Each parking space is about 25 ft long and has a posted 30-minute time limit. Observations indicate that each parking space is used by about two vehicles per hour. The parking lane extends along Maple Avenue to within 5 ft of the intersection with Market Street. Buildings along Maple Avenue are close to the curb, having a setback of 10 ft from the edge of pavement. This narrow setback distance may make it unrealistic to consider widening Maple Avenue to provide additional traffic lanes at the intersection.

Define Problem and Cause

Define Problem and Identify Cause. Based on the results of the observational study, it was found that sufficient evidence existed to justify continuing with the engineering study. Using the information provided in Table 2-1 of the *ESG*, possible causes for the observed problems were identified. This information is summarized in Table A-24.

Stage 1: Step 2. Select Candidate Alternatives

Identify Potential Alternatives

Table 2-2 in the *ESG* was consulted to determine potential alternatives that could address the observed problems at the intersection. The alternatives identified in Table 2-2 are summarized in Table A-25.

Organize and Select Alternatives

The potential alternatives were subjected to a preliminary screening to eliminate any alternatives that would not be feasible at the site. Judgment was used to identify those alternatives that were clearly not feasible due to site-specific constraints. Based on this screening, “peak-hour parking prohibition” was eliminated in preference to “permanent parking prohibition” due to observed delays during both peak and off-peak hours. Permanent prohibition of on-street parking would only be applicable to the intersection influence area to minimize the adverse effect of parking loss on local merchants. An increase in right-turn radius was eliminated in preference to the “add second lane” alternative due to the extent of observed traffic queues. The roundabout alternative was eliminated due to right-of-way limitations. Finally, left-turn prohibition was eliminated because the city council had previously determined that this alternative was not acceptable in the downtown area.

TABLE A-25 Case Study 3: Potential alternatives

Possible Cause	Corrective Strategy	Potential Alternatives
Excessive on-street parking activity on minor road.	Reduce activity.	1. Prohibit on-street parking during peak hours. **2. Prohibit on-street parking permanently.**
Inadequate separation of minor-road movements.	Separate conflicting flows.	**1. Add second lane on minor road.** 2. Increase right-turn radius.
Inadequate capacity for left-turn movements.	Increase capacity.	**1. Convert to traffic signal control.** 2. Convert to roundabout.
	Reduce demand.	1. Prohibit left-turns during peak hours with signing. 2. Prohibit left-turns permanently with channelization.

Based on this analysis, the following alternatives were found to merit further study:

- Add second lane on minor street (by removing on-street parking near the intersection) and
- Convert to traffic signal control.

Stage 1: Step 3. Select Viable Alternatives

The candidate alternatives selected in Step 2 were more closely examined in this step. This examination focused on evaluating each alternative to assess its "viability" (i.e., its ability to address the observed problems effectively).

Gather Information

Identify Data. Table 2-5 in the *ESG* was used to identify the data needed to evaluate the candidate alternatives. Based on the information in this table, it was determined that the following data were needed for guideline evaluation:

- Major- and minor-road turn movement volume (8 hours),
- Heavy vehicle volume (8 hours),
- Pedestrian volume (4 hours),
- Gap frequency (4 hours),
- 85th percentile approach speed,
- ✓ Area population,
- ✓ Number of lanes, and
- ✓ Crash history by type.

Of these data, information about the last three items had already been gathered in a previous step. The minimum number of hours for which data would be collected is indicated in the list above.

Collect Data. At the onset of this task, it was noted that vehicular volumes and approach speed were common to most of the alternatives. It was also noted that the satisfaction of one signal warrant would confirm the viability of the signal alternative. From this assessment, it was decided initially to collect only volume and speed data. These data would allow a check of the *Manual on Uniform Traffic Control Devices-Millennium Edition* (i.e., *MUTCD 2000*) (*A-2*) Signal Warrants 1 and 2. If these warrants were not satisfied, then Warrants 4 and 8 would be checked.

Warrant 3 was not evaluated because the intersection was not believed to represent an "unusual case." Specifically, the intersection was not near an office complex, manufacturing plant, or other facility that generated high volumes over a short period.

Traffic volumes were measured during 8 hours on a Monday in October. These counts were adjusted for weekly and monthly variations to yield an estimate of the average-day volume. The morning peak hour, off-peak hour, and afternoon peak hour were found to occur from 7:00 to 8:00 a.m., 1:00 to 2:00 p.m., and 4:30 to 5:30 p.m., respectively. The turn movement volumes for these hours are provided in Table A-26.

During the study, heavy vehicles were found to comprise about 2 percent of all traffic entering the intersection. Pedestrian volumes were constant throughout the day with about 10 ped/hr crossing Maple Avenue and 15 ped/hr crossing Market Street (total of all crossings on both corners). A spot speed study was conducted for Market Street; the 85th percentile speed was found to be 34 mph.

Assess and Select Alternatives

Convert to Traffic Signal Control. The traffic signal warrants in the *MUTCD 2000* (*A-2*) were evaluated to assess the viability of the traffic signal alternative. Based on this evaluation, it was determined that Warrant 2: Four-Hour Vehicular Volume was satisfied. It should be noted that the left-turn bay on the westbound approach was included in the count of major-street lanes (i.e., 2 lanes) for the following reasons: (1) the approach had only one lane, and (2) the left-turn volume was about one-half of the total approach volume.

To complete the warrant check, the presence of conditions that could misdirect the signal warrant check was assessed. A review of Table 2-15 in the *ESG* revealed that two conditions exist that could affect the warrant analysis conclusions. One condition relates to the fact that the intersection has only three approach legs. The second condition relates to the significant number of right-turn vehicles on the minor-street approach.

TABLE A-26 Case Study 3: Turn movement volumes

Hour	Volume (veh/h)							
	Northbound		Minor Total	Eastbound		Westbound		Major Total
	Left	Right		Through	Right	Left	Through	
7:00 - 8:00 a.m.	100	145	245	126	145	209	244	724
1:00 - 2:00 p.m.	164	191	355	138	210	189	183	720
4:30 - 5:30 p.m.	126	340	466	308	197	288	213	1,006

The discussion associated with Table 2-15 indicated that the *MUTCD's* volume-based warrants might have threshold values that are too low for a three-leg intersection. Based on the guidance in Chapter 2 of the *ESG*, the major and minor roads at a three-leg intersection may need to have volumes of 1,000 and 200 veh/h, respectively, for the 8 highest hours before a traffic signal is likely to be justified. A check of the volumes at the subject intersection indicated that this "1,000/200 veh/h" criterion was *not* met.

The impact of minor-street right-turn volume on the accuracy of the warrant check was also investigated. Figure 2-11 in the *ESG* was consulted to determine the extent to which the right-turn volume might be reduced in the warrant check. Based on this examination, it was found that the minor-street right-turn volume probably should not be included in the warrant analysis (except during the afternoon peak hour). In fact, when Warrant 2 was re-evaluated without this right-turn volume, it was *not* satisfied.

In summary, the warrant analysis indicated that a traffic signal might improve traffic operations at the intersection. However, guidelines in Chapter 2 of the *ESG* indicated that the three-leg geometry and ample right-turn capacity of the intersection might have an impact on the accuracy of the warrant check conclusions. It was determined that the operation of the traffic signal alternative would be carefully examined during the engineering study stage to confirm the benefits of signalization relative to the other alternative.

Add Second Lane on Minor Street. Figure 2-4 in the *ESG* was examined to determine whether adding an exclusive right-turn bay to Maple Avenue would improve traffic operations at the intersection. This bay would be created by removing some on-street parking on the east side of Maple Avenue. The evaluation was based on a right-turn volume that represented 73 percent of the approach traffic stream during the afternoon peak hour. Based on this examination, it was found that a right-turn bay could improve traffic operations during the peak traffic demand hours.

Summary of Viable Alternatives

Based on the assessment of viable alternatives, the candidate alternatives identified in Step 2 were selected for more detailed analysis in the next stage of the assessment process. These alternatives are summarized below:

- **TWSC:** Base case (existing intersection with no improvements);
- **Signal:** Conversion to traffic signal control; and
- **Bay:** Stop control with right-turn bay on the minor street (i.e., Maple Avenue).

The designation shown in bold type is used in subsequent sections to refer to the various alternatives.

STAGE 2: ENGINEERING STUDY

The operational performance of the alternatives identified in the alternative identification and screening stage were evaluated in the engineering study stage. This stage consists of three steps:

1. Determine study type,
2. Select analysis tool, and
3. Conduct evaluation.

Stage 2: Step 1. Determine Study Type

The first step of the engineering study stage required a determination of the type of study needed. Initially, the relationship between the subject intersection and any upstream or downstream signalized intersection was evaluated. Then, the analysis detail required for the assessment of each alternative was determined.

Determine Type of Operation

Figure 3-1 of the *ESG* was consulted to determine whether the subject intersection was isolated from the operation of the upstream intersections. This figure can be used to determine if an intersection is isolated based on consideration of the distance to the nearest signalized intersection and the two-way traffic volume. The nearest signalized intersection in each direction of travel is ½ mile from the subject intersection. The afternoon peak-hour two-way volumes on Maple Avenue, Market Street (west leg), and Market Street (east leg) are 951, 844, and 1,149 veh/h, respectively. Based on this information and the guidelines in Figure 3-1, it was concluded that the subject intersection was effectively isolated from nearby signalized intersection operations.

Determine Type of Evaluation

Next, the intersection was examined to determine whether a formal evaluation of alternatives was required. The *ESG* suggests that an informal evaluation (i.e., implementation and field study) is possible when an intersection has only one viable alternative. However, the fact that the subject intersection is associated with several viable alternatives required the conduct of a formal evaluation.

Stage 2: Step 2. Select Analysis Tool

During this step, an analysis tool was selected for evaluating the operation of the subject intersection and its alternatives. This selection was based on an identification of the desired analysis tool capabilities and a comparison of this list with the actual capabilities of the available tools. The tool selected for use was the one that provided all of the desired capabilities.

Identify Desired Capabilities

Table 3-1 in the *ESG* was examined to determine which analysis tool capabilities would be required for this evaluation. Based on this examination, it was determined that the analysis tool selected must be able to evaluate all of the traffic control modes used with the alternatives (i.e., two-way stop control and actuated traffic signal control). It was also determined that the analysis tool must be able to evaluate left- and right-turn bays, shared-lane approaches, and on-street parking activity. Finally, it was decided that the analysis tool should provide a level-of-service indication.

Evaluate and Select Analysis Tool

Three analysis tools were available for the evaluation: HCS (version 3.1c), CORSIM (version 4.3), and SIDRA (version 5.02a). Each tool was evaluated for its ability to provide the desired capabilities. Based on this evaluation, CORSIM was determined to be the only analysis tool that could explicitly model parking maneuver frequency and duration on a stall-by-stall basis.

It was noted that the version of CORSIM used did not provide an estimate of control delay (as would be needed to determine level of service). However, previous efforts by city engineers to validate this tool had revealed that CORSIM's "queue delay" estimate was within 5 percent of the control delay (based on field measurements). Therefore, the "queue delay" obtained from CORSIM was used for the evaluation with the recognition that any level-of-service estimate would be approximate.

Stage 2: Step 3. Conduct Evaluation

The final step in the engineering study stage was to determine the operational performance of the viable alternatives. Before conducting this evaluation, the need for additional information and field data was reviewed. Then, the operational performance of each alternative was quantified and its relative effectiveness was determined.

Gather Information

The need for additional data was assessed at this point in the study. This assessment focused on the input data requirements of the chosen analysis tool (i.e., CORSIM). The initial data collection effort was determined to be thorough enough that all necessary data were available. Based on this finding, it was concluded that additional data did not need to be gathered.

Evaluate Operational Performance

Design Alternatives. Some preliminary design decisions were made regarding the operation and geometry of the alternatives before they were evaluated. Most of this effort was devoted to the development of a design and timing plan for the traffic signal control alternative. However, some effort was also expended on the design of the right-turn bay.

Signal Alternative. The signal design process included a determination of the intersection geometry and its signal timing. Guidelines in Chapter 3 of the *ESG* were used for this purpose. In particular, guidelines regarding the need for right-turn lanes are provided in Figure 3-3. A check of these guidelines indicated that a northbound right-turn lane was not needed for the signal alternative.

Guidance regarding the operation of the signal controller is provided in Table 3-5 of the *ESG*. This guidance indicated that actuated operation is appropriate for isolated intersections.

Guidance regarding the need for left-turn phasing is provided in Figure 3-5 of the *ESG*. A check of the information in this figure indicated that a protected-permitted left-turn phase was needed for the westbound left-turn movement during the afternoon peak hour.

Traffic volumes recorded for the afternoon peak hour were used to determine the controller settings. This analysis followed the guidelines associated with Tables 3-7 and 3-8 in the *ESG*. The settings determined from this analysis were reasoned to be sufficiently accurate for the engineering study evaluation; however, it was recognized that they may need to be refined if the signal alternative is selected.

Bay Alternative. Table 2-13 of the *ESG* provided some information regarding the appropriate length of a right-turn bay on the minor street. According to this table, a right-turn bay should be long enough to provide for right-turn vehicle storage. Table A-26 indicated that the conflicting volume during the afternoon peak hour is 407 veh/h (=308 + 197/2). Extrapolation of Figure 2-8 of the *ESG* for a right-turn volume of 340 veh/h indicated that the right-turn bay on the minor street should be at least 260 ft long during the peak hour. However, due to the desires of local merchants to maintain as much on-street parking as possible and the fact that a 50-ft bay would be adequate for all other hours, it was determined that a reasonable maximum bay length was 100 ft. The use of this length required the permanent removal of four parking spaces.

Evaluate Alternatives. Because a stochastic simulation model (i.e., CORSIM) was being used, the minimum simulation period had to be determined so that statistically reliable results could be obtained. This period was established by considering the minimum simulation period based on the individual movement delays and the average intersection delay. Guidelines in Chapter 3 of the *ESG* were used to

TABLE A-27 Case Study 3: Minimum simulation period evaluation

Time Period	Individual Movement Delay Basis						Avg. Int. Delay Basis		Simulation Period Used (h)
	Volume (veh/h) [1]				Minimum Volume (veh/h)	Minimum Simulation Period (h)	Total Volume (veh/h)	Minimum Simulation Period (h)	
	Northbd.	Eastbd.	Westbd.						
	L + R	T + R	L	T					
7:00 - 8:00 a.m.	245	271	209	244	209	**0.8**	969	**0.5**	**1.0**
1:00 - 2:00 p.m.	355	348	189	183	183	**0.9**	1,075	**0.5**	**1.0**
4:30 - 5:30 p.m.	466	505	288	213	213	**0.8**	1,472	**0.4**	**1.0**

Note:
1 - Movement types: L = left-turn, T = through, R = right-turn.

determine the minimum simulation period based on the volume of the individual movements and the total volume entering the intersection.

The minimum simulation period based on individual movement delay was determined first. Traffic volumes recorded for the morning peak, afternoon peak, and off-peak hours were used for this determination. Movements that shared a traffic lane were combined, as suggested in the *ESG*. Figure 3-12 of the *ESG* was used to determine the minimum simulation period needed for each movement volume. The smallest movement volume for a given hour dictated the minimum simulation period based on individual movement delay. The results of this evaluation are shown in Column 7 of Table A-27.

Next, the minimum simulation period based on average intersection delay was determined. The total entering volumes are 969 vehicles for the morning peak, 1,075 vehicles for the off-peak, and 1,472 vehicles for the afternoon peak hour. Figure 3-13 of the *ESG* was used to determine the minimum simulation period needed for the overall intersection. The results of this evaluation are shown in Column 9 of Table A-27.

The last step in this process was to compare the minimum simulation period based on individual movements with that based on the overall intersection. The larger of these two values was selected as the minimum simulation period used for the evaluation. It was recognized that a simulation of this duration would provide the statistical precision needed in all delay estimates. The simulation period used for the evaluation is shown in Column 10 of Table A-27. For each time period, the minimum simulation period was found to be 0.8 or 0.9 h. For simplicity, the minimum value was rounded up to an even 1 hour for all time periods.

CORSIM was used to evaluate each viable alternative for the morning peak, afternoon peak, and off-peak hours. The individual movement delays predicted for the afternoon peak traffic hour are summarized in Table A-28. The Bay alternative (i.e., two-way stop control with right-turn bay on Maple Avenue) resulted in the smallest intersection delay. The existing alternative (TWSC) produced unacceptable delay (i.e., it exceeded the delay threshold associated with LOS D and had more than 4.0 veh-hr of delay) on the minor street. In contrast, the Signal alternative resulted in the largest intersection delay.

The impact of each alternative on average intersection delay was also examined. Figure A-10 illustrates the average intersection delay for each alternative. The trends are consistent with those found in Table A-28 and indicate that the Bay alternative results in the smallest delay for all three analysis periods.

The delay associated with each alternative was tested to determine whether it was significantly different from that of the other alternatives. The comparison technique was based on an examination of the "delay ratio" (i.e., smaller delay divided by larger delay), as described in Chapter 3 of the *ESG*. Ratios less than 0.9 indicate that the two delays (and corresponding alternatives) are significantly different. Table A-29 summarizes the comparison of each pair of alternatives.

The delay ratio for each pair of alternatives was found to be less than 0.9. This finding indicated that the delays asso-

TABLE A-28 Case Study 3: Delay summary for peak traffic hour

Alternative	Movement Delay (s/veh) [1]						Intersection Delay (s/veh)
	Northbound		Eastbound		Westbound		
	Left-turn	Right-turn	Through	Right-turn	Left-turn	Through	
1. TWSC	75	62	1	3	2	0	21
2. Signal	38	32	31	21	14	10	24
3. Bay	21	18	1	1	2	0	6

Note:
1 - The true mean delay for each movement is within ± 10 percent of the value shown (with 90% confidence).

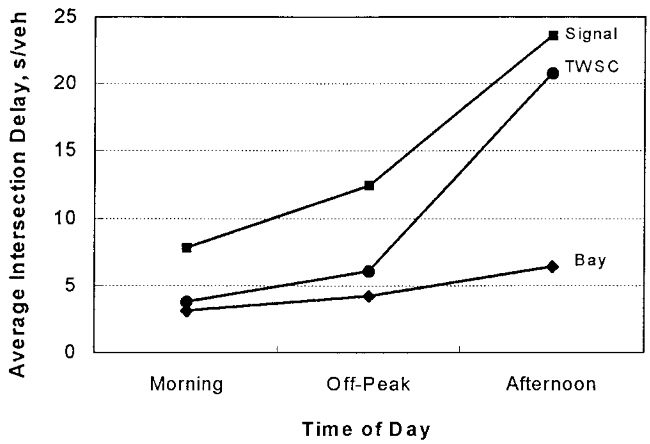


Figure A-10. Case Study 3: Average intersection delay.

ciated with each pair are significantly different (with 90 percent confidence) and that the trends shown in Figure A-10 are very likely to be accurate. Most important, the analysis confirms that the Bay alternative is the best overall alternative in terms of traffic operations.

Determine Alternative Effectiveness

The findings from the operations analysis were evaluated to determine whether each alternative served traffic effectively. This determination was made based on the definitions of "acceptable level of service" and "acceptable operation" defined in Chapter 3 of the *ESG*. Table A-30 summarizes the effectiveness of each alternative.

As suggested by the response provided in the last column of Table A-30, only one alternative was found to satisfy the "effectiveness" criteria. Specifically, the Bay alternative (i.e., the addition of a right-turn bay on the minor street by removing 100 ft of parking) was the only alternative that satisfied all three criteria. It reduced the average intersection delay to acceptable levels and provided minor-street delays of 4 veh-h or less during the peak hour.

STAGE 3: ALTERNATIVE SELECTION

During the alternative selection stage, the alternatives advanced from the engineering study stage are reviewed for their potential impact on traffic operations, safety, and the environment. The alternative selection stage consists of the following three steps:

1. Identify impacts,
2. Select best alternative, and
3. Document study.

Stage 3: Step 1. Identify Impacts

In general, factors are typically identified that might influence alternative selection. These decision factors include traffic operations, traffic safety, direct cost, environment, and aesthetics. In this case, an alternative's impact on local business was also relevant.

An evaluation of the impact of the Bay alternative (i.e., a right-turn bay on the minor-street approach) indicated that traffic operations and safety at the intersection would be greatly improved for a nominal direct cost (i.e., the cost of delineating a right-turn bay and removing parking signs). However, the indirect cost to local businesses through the removal of four parking slots was difficult to quantify. It was noted that convenient customer parking would still be available within 150 ft of the affected businesses. It was also thought that the improved operation of the intersection may bring some customers back to area businesses. In summary, the cost of parking removal was reasoned to be acceptable, given the benefits received directly by motorists and indirectly by the area businesses in terms of increased business activity.

TABLE A-29 Case Study 3: Analysis of delay reduction associated with selected alternatives

Time Period	Average Intersection Delay		Delay Ratio	Reduction is Significant?	Better Alternative
	Alternative 1	Alternative 2			
Morning Peak	TWSC	Signal	0.49	Yes	TWSC
	Bay	TWSC	0.82	Yes	Bay
	Bay	Signal	0.40	Yes	Bay
Mid-Day Peak	TWSC	Signal	0.49	Yes	TWSC
	Bay	TWSC	0.69	Yes	Bay
	Bay	Signal	0.34	Yes	Bay
Afternoon Peak	TWSC	Signal	0.86	Yes	TWSC
	Bay	TWSC	0.31	Yes	Bay
	Bay	Signal	0.27	Yes	Bay

TABLE A-30 Case Study 3: Alternative effectiveness

Alternative	Reduces Average Intersection Delay?	Acceptable Level of Service?	Acceptable Operation?	Effective Alternative?
1. TWSC	n.a.	Yes	No	No
2. Signal	No	Yes	Yes	No
3. Bay	Yes	Yes	Yes	Yes

Note:
n.a. - not applicable.

Stage 3: Step 2. Select Best Alternative

Based on this analysis, the Bay alternative (i.e., a right-turn bay on the minor-street approach) was selected as the best overall alternative.

Stage 3: Step 3. Document Study

A report documenting the results of the engineering study was prepared and submitted.

CASE STUDY 4

SYNOPSIS

This case study illustrates the application of the *Engineering Study Guide* (*ESG*) to an intersection being planned for construction on a busy arterial street. The location of the proposed intersection is very near to a signalized intersection. The potential for queue spillback from the signalized intersection into the subject intersection is high and would create excessive delay if it occurred. The CORSIM simulation model was used to analyze the operation of the proposed intersection and its two nearby signalized intersections. The objective was to resolve queuing problems and minimize the impact of the new intersection on arterial operations.

BACKGROUND

Main Street is a busy arterial street that provides for a significant amount of east-to-west intracity travel. A developer is proposing to build an office park on the south side of Main Street between Rio Road and Sunset Boulevard. The developer is also proposing to fund the extension of a city street (i.e., Elm Street) through the property to provide a point of access to the arterial street system. The proposed intersection would have three legs with Elm Street representing the minor street. It would be located about 400 ft west of Sunset Boulevard and 800 ft east of Rio Road.

The City Traffic Engineer is concerned that projected Elm Street traffic demands (at the intersection with Main Street) would exceed the capacity of a two-way stop-controlled intersection. There is also a concern that the projected demands may justify a traffic signal and that this signal may make it difficult to maintain good traffic progression along Main Street. Finally, there is a concern that the intersection's proposed location is too close to the signalized intersection at Sunset Boulevard. Queues from this intersection may spill back into and block traffic at the new intersection. Because of these concerns, an engineering study was undertaken to evaluate the feasibility of adding a new intersection to Main Street.

Main Street is major arterial in a city of 100,000 people. The intersections of Main Street with Rio Road and Sunset Boulevard are signalized and are coordinated with one another. The timing plan for these two intersections was developed about 3 years ago. Traffic progression could be significantly improved if the timing plan were updated. All streets have a posted speed limit of 35 mph. Figure A-11 illustrates the proposed location of the Elm Street intersection. It also shows the two signalized intersections that are adjacent to Elm Street.

STAGE 1: ALTERNATIVE IDENTIFICATION AND SCREENING

The first stage of the engineering assessment process involved diagnosing the problem at the intersection and determining potential corrective measures. Candidate alternatives were identified and given a preliminary screening to determine whether they represented viable solutions to the problem. The alternative identification and screening stage consists of the following steps:

1. Define problem and cause,
2. Select candidate alternatives, and
3. Select viable alternatives.

Stage 1: Step 1. Define Problem and Cause

Gather Information

The first step was to gather information about the arterial. Information about the proposed intersection was also collected; however, much of the information available was based on traffic projections and the proposed site layout. Information was also gathered from agency records and a visit to the site. The data gathered during this step are summarized in the remainder of this section.

Historic Data. The last engineering study of the arterial was a corridor study conducted about 10 years ago. This study evaluated the arterial's ability to serve intracity travel and made recommendations about improvements needed to improve arterial capacity. Regional traffic counts recorded within the past year indicated that the annual average daily traffic (AADT) on Main Street is 29,400 vehicles per day.

Observational Study. The arterial segment was visited during a typical weekday to determine the character of traffic flow, the extent of the traffic queues, and the geometry of the intersections. The observational study revealed that traffic queues from the intersection of Main Street and Sunset Boulevard extend 450 ft back (toward Rio Road) during the afternoon peak traffic hour. Observations indicate that this queue length could be reduced by improved signal timing at Sunset Boulevard.

Site Survey. A site survey was also conducted during the site visit. Key geometric features of the arterial noted during this survey are summarized in Figure A-11.

Traffic Projections. The developer was asked to define the characteristics of the proposed business park (e.g., number

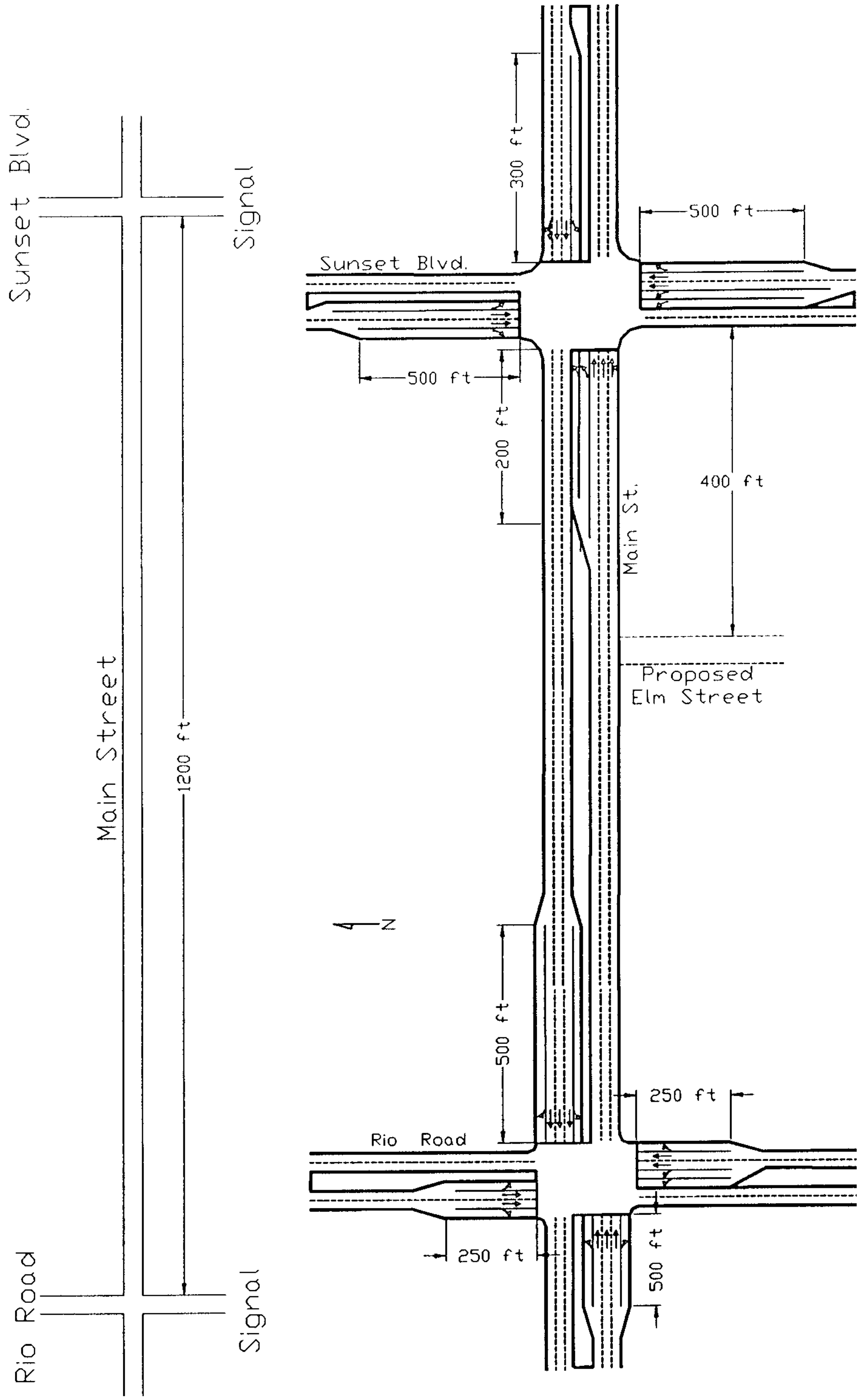


Figure A-11. Case Study 4: Existing arterial geometry (not to scale).

TABLE A-31 Case Study 4: Potential problems and causes

Potential Problem	Possible Cause
Excessive delay to minor-road movements (on Elm Street).	• Inadequate capacity for minor-road movements. • Arrival of major-road platoons is staggered such that there are limited opportunities to enter intersection. • Queue spillback from downstream signal.

of employees, types of business, and square footage). These data were then used to develop an estimate of the weekday traffic demand on Elm Street. Based on this analysis, the AADT for Elm Street was estimated as 9,200 vehicles per day.

Define Problem and Cause

Assess Evidence. The City Traffic Engineer's concerns appear to have merit based on the findings of the observational study. Possible problems that might occur from the construction of the proposed intersection would include (1) excessive delay on the Elm Street approach and (2) disruption to traffic progression if the Elm Street intersection were signalized. Also, two operational problems were found on Main Street. One problem was excessive delay and queuing on the eastbound approach to the intersection of Main Street and Sunset Boulevard A second problem was poor traffic progression along Main Street. Based on the results of the observational study, it was found that sufficient evidence existed to justify continuing with the engineering study.

Define Problem and Identify Cause. Using the information provided in Table 2-1 of the *ESG*, possible causes for the aforementioned problems were identified. This information is summarized in Table A-31.

Stage 1: Step 2. Select Candidate Alternatives

Identify Potential Alternatives

Tables 2-2 and 2-3 in the *ESG* were consulted to determine potential alternatives that could address the observed problems at the intersection. The alternatives identified in these tables are summarized in Table A-32. Two-way stop control was added to the list of potential alternatives to represent the "base" control mode because it represented the least-restrictive mode that could be used at an arterial-collector intersection.

Organize and Select Alternatives

The potential alternatives were subjected to a preliminary screening to eliminate any alternatives that would not be feasible at the site. Judgment was used to identify those alternatives that were clearly not feasible due to site-specific constraints. Table A-33 lists the alternatives eliminated and provides the reason for this action.

Based on this analysis, the following alternatives were found to merit further study:

- Use two-way stop control (at Elm Street);
- Convert to traffic signal controlled coordinated system (at Elm Street);
- Adjust signal timing at downstream signal;

TABLE A-32 Case Study 4: Potential alternatives

Possible Cause	Corrective Strategy	Potential Alternatives
Inadequate capacity for minor-road movements.	Increase approach capacity.	1. Convert to roundabout. 2. Convert to yield control. **3. Convert to traffic signal control.** 4. Convert to multi-way stop control.
Queue spillback from downstream signal.	Increase downstream capacity.	**1. Adjust signal timing at downstream signal.** **2. Modify signal coordination.** **3. Add traffic lanes at downstream signal.**
	Separate conflicts.	1. Relocate subject intersection.
	Provide guidance.	1. Add advisory signing.
Arrival of major-road platoons is staggered such that there are limited opportunities to enter intersection.	Modify arrival patterns.	**1. Adjust signal timing at upstream signals.** 2. Relocate subject intersection.
	Concentrate and organize platoons.	**1. Convert to traffic signal (coordinated system).**
--	--	**1. Two-way stop-control (base control mode).**

TABLE A-33 Case Study 4: Alternatives eliminated

Alternative	Reason for Elimination
Convert to roundabout.	It is agency policy not to install roundabouts on coordinated arterial streets.
Convert to yield control.	Not applicable to intersections with high-volume arterials.
Convert to multi-way stop control.	Based on the engineer's judgment, multi-way stop control is not an appropriate form of traffic control for a major arterial street.
Relocate subject intersection.	The intersection could not be relocated without acquiring some very expensive existing businesses on Main Street.
Add advisory signing.	Based on the engineer's judgment, advisory signing is unlikely to solve the problem.

- Modify signal coordination (for all signalized intersections on Main Street); and
- Add traffic lanes at downstream signal (on Main Street at Sunset Boulevard).

Stage 1: Step 3. Select Viable Alternatives

The candidate alternatives selected in Step 2 were more closely examined in this step. This examination focused on evaluating each alternative to assess its "viability" (i.e., its ability to address the observed problems effectively).

Guidelines in Chapter 2 of the *ESG* were used to assess the viability of traffic signal control at Elm Street. Similar guidelines were not available for assessing the viability of signal timing adjustment, signal coordination, and adding traffic lanes to downstream signalized intersections. Thus, these alternatives were advanced to the engineering study stage for a formal evaluation.

Gather Information

Identify Data. Table 2-5 in the *ESG* was used to identify the data needed to evaluate the candidate alternatives. Based on the information in this table, it was determined that the following data were needed for guideline evaluation:

- Major- and minor-road approach volume for proposed intersection (8 hours),
- Heavy vehicle volume for proposed intersection (8 hours),
- 85th percentile approach speed (on Main Street),
- Progression quality (on Main Street),
- ✓ Area population, and
- ✓ Number of lanes for proposed intersection.

These data are sufficient to evaluate the *Manual on Uniform Traffic Control Devices-Millennium Edition* (i.e., *MUTCD 2000*) (*A-2*) Signal Warrants 1, 2, 3, and 6. Information about the last two items had already been gathered in a previous step. The minimum number of hours for which the data were collected is indicated in the list above.

Collect Data. At the onset of this task, it was noted that the satisfaction of one signal warrant would confirm the viability of the corresponding alternative. From this assessment, it was decided to base the warrant check on volume and speed data. These data would allow a check of Warrants 1, 2, and 3. If none of these warrants were satisfied, then Warrant 6 would be evaluated.

To facilitate the warrant check for the proposed intersection, an 8-hour count was synthesized using a combination of forecast and existing traffic data. The objective of this approach was to derive a reasonably accurate estimate of the combined traffic volumes for each of the 8 highest hours of the average day. The forecast AADT for Elm Street (including development-related and external through traffic) was converted to hourly traffic volume estimates using the procedures described in Appendix C. Main Street traffic volumes were obtained from an 8-hour manual count.

A spot speed study was conducted for Main Street; the 85th percentile speed was found to be 37 mph. Heavy vehicles were noted to comprise about 2 percent of the traffic stream.

Assess and Select Alternatives

The traffic signal warrants in the *MUTCD 2000* (*A-2*) were evaluated to assess the viability of the traffic signal alternative. The results of this evaluation are summarized in Table A-34. Based on this evaluation, it was determined that Warrants 1, 2, and 3 were satisfied. It should be noted that the Elm Street approach was assumed to have a minimum of two traffic lanes based on the magnitude of the approach volume.

To complete the warrant check, there was an assessment of possible conditions that could misdirect the signal warrant check. A review of Table 2-15 in the *ESG* revealed that progressive traffic flow on the major street could affect the warrant analysis conclusions. Discussion associated with Table 2-15 indicated that dense traffic platoons can affect operations at the proposed intersection to the extent that conclusions reached from the warrant analysis may be inaccurate. Based on this analysis, it was concluded that the operation of the traffic signal alternative would be carefully examined during the engineering study stage to confirm the benefits of signalization.

TABLE A-34 Case Study 4: Signal warrant check summary

Volume Category	Estimated Volumes		Applicable Warrant	Warrant Volumes (veh/h)		Warrant Satisfied?
	Major (two-way)	Minor (highest app.)		Major (two-way)	Minor (highest app.)	
AADT (veh/d)	29,400	4,000				
1st highest hour (veh/h)	2,350	320	3	>1,700	150	Yes
4th highest hour (veh/h)	2,050	280	2	>1,300	115	Yes
8th highest hour (veh/h)	1,760	240	1 A	600	200	Yes
			1 B	900	100	Yes

Note:
1 - Warrant check based on three lanes on each major-street approach and two lanes on the minor-street approach.

Summary of Viable Alternatives

The candidate alternatives selected in Step 2 were determined to be viable alternatives and suitable for formal evaluation in the engineering study stage. However, these alternatives were reconfigured to form alternatives that more comprehensively addressed the observed operational problems. The revised alternatives follow:

1. **Base:** Two-way stop control at Elm Street intersection with existing signal timings;
2. **Bay:** Two-way stop control at Elm Street intersection, adjust timing and coordinate existing signals at Rio Road and at Sunset Boulevard, and add a right-turn bay on eastbound Main Street at its intersection with Sunset Boulevard; and
3. **Signal:** Convert to traffic signal control at Elm Street intersection, adjust timing and coordinate all signalized intersections, and add a right-turn bay on eastbound Main Street at its intersection with Sunset Boulevard

The designation shown in bold type is used in subsequent sections to refer to the various alternatives.

STAGE 2: ENGINEERING STUDY

The operational performance of the alternatives identified in the alternative identification and screening stage were evaluated in the engineering study stage. This stage consists of three steps:

1. Determine study type,
2. Select analysis tool, and
3. Conduct evaluation.

Stage 2: Step 1. Determine Study Type

The first step of the engineering study stage required a determination of the type of study needed. Initially, the relationship between the subject intersection and any upstream or downstream signalized intersection was evaluated. Then, the analysis detail required for the assessment of each alternative was determined.

Determine Type of Operation

Figure 3-1 in the *ESG* was consulted to determine whether the proposed intersection would be isolated from the operation of upstream intersections. This figure can be used to determine if an intersection is isolated based on consideration of the distance to the nearest signalized intersection and the two-way traffic volume. The nearest intersection to the east (i.e., Sunset Blvd.) is 400 ft from the proposed intersection. The nearest intersection to the west (i.e., Rio Road) is 800 ft from the proposed intersection. The peak hour two-way volume on both street segments is about 2,700 veh/h. Based on this information and the guidelines in Figure 3-1, it was concluded that the intersection and all its alternatives would be treated as "non-isolated."

Determine Type of Evaluation

Next, the intersection was examined to determine whether a formal evaluation of alternatives was required. The *ESG* suggests that an informal evaluation (i.e., implementation and field study) is possible when an intersection is isolated and has only one viable alternative. However, a formal evaluation was required in this case, because the subject intersection was classified as "non-isolated."

Stage 2: Step 2. Select Analysis Tool

During this step, an analysis tool was selected for evaluating the operation of the subject intersection and its alternatives. This selection was based on an identification of the desired analysis tool capabilities and a comparison of this list with the actual capabilities of the available tools. The tool selected for use was the one that provided all of the desired capabilities.

Identify Desired Capabilities

Table 3-1 in the *ESG* was examined to determine which analysis tool capabilities would be required for this evaluation. Based on this examination, it was determined that the analysis tool selected must be able to evaluate all of the traffic control modes used with the alternatives (i.e., two-way stop control and signal coordination). It was also determined that the analysis tool must be able to evaluate left- and right-turn bays, right-turn-on-red at signalized intersections, and spillback from a downstream signal. Finally, it was decided that the analysis tool should provide a level-of-service (LOS) indication.

Evaluate and Select Analysis Tool

Three analysis tools were available for the evaluation: HCS (version 3.1c), CORSIM (version 4.3), and TRANSYT-7F (release 8). Each tool was evaluated for its ability to provide the desired capabilities. Based on this evaluation, CORSIM was determined to be the only analysis tool that could fully evaluate the effect of queue spillback on intersection operations.

It was noted that the version of CORSIM used did not provide an estimate of control delay (as would be needed to determine LOS). However, previous efforts by city engineers to validate this tool had revealed that CORSIM's "queue delay" estimate was within 5 percent of the control delay (based on field measurements). Therefore, the "queue delay" obtained from CORSIM was used for the evaluation with the recognition that any LOS estimate would be approximate.

Stage 2: Step 3. Conduct Evaluation

The final step in the engineering study stage was to determine the operational performance of the viable alternatives. Before conducting this evaluation, the need for additional information and field data was reviewed. Then, the operational performance of each alternative was quantified and its relative effectiveness was determined.

Gather Information

The need for additional data was assessed at this point in the study. This assessment focused on the input data requirements of the chosen analysis tool (i.e., CORSIM). The following data were identified as necessary for the evaluation:

- Major- and minor-street turn movement volumes for all intersections;
- Signal phase sequence and interval duration for both signalized intersections; and
- Relative offset between signals.

It was determined that the evaluation would include the morning and afternoon peak traffic hours as well as one mid-day off-peak hour. Turn movement counts were recorded on a Tuesday in July at the two signalized intersections. These counts were adjusted for weekly and monthly variations (using the procedure described in Appendix C) to obtain an estimate of the average-day volume. Turn movement volumes for the proposed intersection were derived using typical turn movement percentages for similar arterial/collector intersections in the vicinity. The turn movement volumes used in the evaluation are summarized in Table A-35. Signal phase sequence, interval duration, and relative offset were obtained from agency signal timing records.

Evaluate Operational Performance

Design Alternatives. Some preliminary design decisions were made regarding the operation and the geometry of the

TABLE A-35 Case Study 4: Turn movement volumes

Time Period	Intersecting Street	Volume (veh/h) [1]											
		Northbound			Southbound			Eastbound			Westbound		
		L	T	R	L	T	R	L	T	R	L	T	R
AM Peak	Rio Road	340	162	86	158	414	199	180	653	350	180	1422	142
	Elm Street	109	0	128	0	0	0	0	771	126	264	1635	0
	Sunset Blvd.	354	578	256	130	398	403	174	518	207	180	1142	190
Off - Peak	Rio Road	252	302	141	238	396	173	180	894	184	112	987	90
	Elm Street	94	0	108	0	0	0	0	1075	198	186	1095	0
	Sunset Blvd.	458	622	387	161	522	325	115	533	535	232	498	142
PM Peak	Rio Road	245	194	89	113	330	293	206	1132	360	240	654	169
	Elm Street	182	0	138	0	0	0	0	1105	229	183	881	0
	Sunset Blvd.	431	618	372	190	635	213	312	439	492	287	420	203

Note:
1 - Movement types: L = left-turn, T = through, R = right-turn.

alternatives before they were evaluated. Most of this effort was devoted to the development of an intersection design and timing plan for the traffic signal control alternative. However, some effort was also expended in the design of the right-turn bay at Sunset Boulevard.

Signal Alternative. The signal design process included a determination of the intersection geometry and its signal timing. Three through lanes were assumed for the major-street approaches to be consistent with the cross section of Main Street.

Guidelines in Chapter 3 of the *ESG* indicated that an exclusive left-turn lane (or bay) may be needed when the left-turn volume exceeds 100 veh/h. The left-turn volume on the westbound approach satisfies this criterion for the peak and off-peak hours. Thus, a left-turn bay was included in the intersection design for the Signal alternative.

Guidelines regarding the need for right-turn lanes are provided in Figure 3-3 of the *ESG*. These guidelines indicated that a right-turn lane was not needed on the eastbound approach. They also indicated that a right-turn lane was not needed on the northbound approach. However, it was decided that two lanes would be provided on the northbound approach to minimize the proposed intersection's impact on arterial progression. These two lanes would include a bay for the left-turn movement and a lane for the right-turn movement.

Guidance regarding the operation of the signal controller is provided in Table 3-5 of the *ESG*. This guidance indicated that pretimed operation is appropriate for intersections having "heavy and consistent" traffic demands. As these characteristics were consistent with traffic demands at the proposed intersection, pretimed operation was assumed for the signal design.

Guidance regarding the need for left-turn phasing is provided in Figure 3-5 of the *ESG*. A check of the information in this figure indicated that left-turn phasing was needed on the major street. Thus, the signal phase sequence consisted of three phases: one for the westbound left-turn and through movements, one for the east and westbound through movements, and one for the northbound left- and right-turn movements.

Traffic volumes recorded for the morning peak, afternoon peak, and mid-day off-peak periods were used to determine the pretimed controller settings. This analysis followed the guidelines associated with Tables 3-7 and 3-8 in the *ESG*. The settings determined from this analysis were reasoned to be sufficiently accurate for the engineering study evaluation; however, it was recognized that they might need to be refined if the signal alternative is selected.

Finally, the cycle length and offset of the signal system were determined using guidelines provided in Chapter 3 of the *ESG*. Guidelines associated with Figure 3-10 indicated that a 50-s cycle length was needed to provide good two-way progression for the existing two signalized intersections (based on a 1,200-ft signal spacing and 35-mph speed). The relative offset between the two existing signals would be equal to the travel time of 25 s. The proposed intersection would have an offset of 0.0 s relative to one of the adjacent signals. It was decided to reference this 0.0-s offset to the nearer signal of the two (i.e., Sunset Boulevard).

Bay Alternative. Figure 3-3 in the *ESG* was consulted to determine whether an exclusive right-turn bay was appropriate on the Main Street approach at the intersection with Sunset Boulevard. Field observations indicated that a bay would reduce the extensive queues found on the eastbound approach by adding capacity and increasing queue storage. Based on the information in Figure 3-3, it was determined that two through lanes and one right-turn lane would be adequate; however, the peak hour volumes were almost large enough to justify three through lanes and one right-turn lane. Therefore, to maintain continuity along the route, the existing three through lanes were maintained and a right-turn bay was added.

Base Alternative. Figure 2-4 in the *ESG* indicates that a minimum of two lanes are needed for the stop-controlled approach to a two-way stop-controlled intersection. Based on this information, the Base alternative was assumed to have two lanes on the Elm Street approach.

Evaluate Alternatives. Because a stochastic simulation model (i.e., CORSIM) was being used, the minimum simulation period had to be determined so that statistically reliable results could be obtained. This period is established by considering the minimum simulation period based on the individual movement delays and the average intersection delay. Guidelines in Chapter 3 of the *ESG* were used to determine the minimum simulation period based on the volume of the individual movements and the total volume entering the intersection.

The minimum simulation period based on individual movement delay was determined first. Traffic volumes for the morning peak, afternoon peak, and mid-day off-peak hours were used for this determination. Movements that shared a traffic lane were combined, as suggested in the *ESG*. Figure 3-12 of the *ESG* was used to determine the minimum simulation period needed for each movement volume. The smallest movement volume for a given hour dictated the minimum simulation period based on individual movement delay. The results of this evaluation are shown in Column 6 of Table A-36.

Next, the minimum simulation period based on average intersection delay was determined. The total entering volume at the proposed intersection was consistently the smallest volume of the three intersections. This volume is listed in Column 7 of Table A-36. Figure 3-13 of the *ESG* was used to determine the minimum simulation period needed for the overall intersection. The results of this evaluation are shown in Column 8 of Table A-36.

The last step in this process was to compare the minimum simulation period based on individual movements with that

TABLE A-36 Case Study 4: Minimum simulation period evaluation

Time Period	Individual Movement Delay Basis					Average Int. Delay Basis		Simulation Period Used (h)
	Minimum Intersection Volume (veh/h) [1]			Minimum Overall Volume (veh/h)	Minimum Simulation Period (h)	Total Volume (veh/h)	Minimum Simulation Period (h)	
	Rio	Elm	Sunset					
AM Peak	86	109	130	86	**2.0**	3,033	**< 0.5**	**2.0**
Off Peak	112	94	115	94	**2.0**	2,756	**< 0.5**	**2.0**
PM Peak	89	138	190	89	**2.0**	2,718	**< 0.5**	**2.0**

Note:
1 - Movement types: L = left-turn, T = through, R = right-turn.

based on the overall intersection. The larger of these two values was selected as the minimum simulation period used for the evaluation. It was recognized that a simulation of this duration would provide the statistical precision needed in all delay estimates. The simulation period used for the evaluation is shown in Column 9 of Table A-36. To control for time-dependencies in the delay statistic, the 2-hour simulation period was divided into two, 1-hour simulation runs. The delay statistics from both runs were then averaged for subsequent analyses.

Table A-37 summarizes the average delay experienced during the afternoon peak hour at each intersection. The Bay alternative (i.e., two-way stop control at Elm Street, add a right-turn bay, and re-timing the existing signals) offered the lowest overall delay at two of the three intersections. The Base alternative (i.e., two-way stop control at Elm Street) was the only alternative where some traffic movements operated at LOS D or worse (based on a delay threshold of 35 s/veh for unsignalized movements and 55 s/veh for signalized movements).

The impact of each alternative on average intersection delay was examined. Table A-38 lists the average delay at each intersection for each alternative and study period. The data in this table indicate that the Bay alternative tends to offer the lowest delay at the Elm Street and the Sunset Boulevard intersections. The Signal alternative offers the lowest delay at the Rio Road intersection.

Figure A-12 illustrates the total delay experienced within the three-intersection arterial for each alternative during each analysis period. The Bay alternative consistently produces the lowest total delay. The Base alternative consistently produces the largest delays.

Determine Alternative Effectiveness

The findings from the operations analysis were evaluated to determine whether each alternative served traffic effectively. This determination was made based on the definitions of "acceptable level of service" and "acceptable operation" as defined in Chapter 3 of the *ESG*. Table A-39 summarizes the results of this evaluation.

For a movement to have "acceptable operation," its level-of-service (LOS) should be no worse than "D" or its total

TABLE A-37 Case Study 4: Delay summary for peak traffic hour

Inter-section	Alter-native	Movement Delay (s/veh)												Inter-section Delay (s/veh)
		Northbound			Southbound			Eastbound			Westbound			
		L	T	R	L	T	R	L	T	R	L	T	R	
Main & Rio	Base	27	24	5	31	26	4	23	17	3	26	16	3	17
	Bay	21	20	5	22	26	4	22	19	3	29	15	2	16
	Signal	22	20	5	22	26	4	23	18	3	27	12	2	16
Main & Elm	Base	**76**	n.a.	**89**	n.a.	n.a.	n.a.	n.a.	5	10	20	0	n.a.	14
	Bay	18	n.a.	5	n.a.	n.a.	n.a.	n.a.	0	0	1	0	n.a.	2
	Signal	21	n.a.	5	n.a.	n.a.	n.a.	n.a.	19	48	22	6	n.a.	17
Main & Sunset	Base	38	18	2	33	17	2	24	**95**	**60**	26	**60**	37	35
	Bay	25	24	3	24	22	3	26	22	7	28	20	10	18
	Signal	24	24	3	24	22	3	21	34	9	29	21	11	20

Note:
1 - Movement types: L = left-turn, T = through, R = right-turn.
n.a. - not applicable.

TABLE A-38 Case Study 4: Average intersection delay

Intersection	Time Period	Average Intersection Delay by Alternative (s/veh)		
		Base	Bay	Signal
Main Street & Rio Road	Morning Peak	20.9	17.6	**17.1**
	Off Peak	19.7	19.0	**17.3**
	Afternoon Peak	16.6	16.3	**15.6**
Main Street & Elm Street	Morning Peak	**1.4**	1.6	7.8
	Off Peak	5.1	**1.2**	7.5
	Afternoon Peak	13.8	**1.7**	16.5
Main Street & Sunset Blvd.	Morning Peak	21.9	**18.3**	19.4
	Off Peak	27.1	**17.6**	18.9
	Afternoon Peak	35.4	**18.4**	19.5

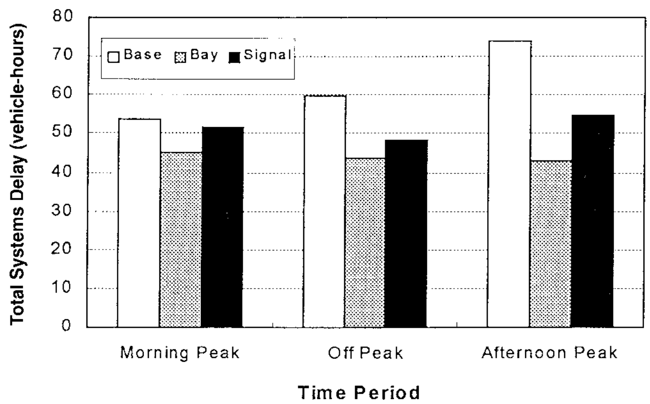


Figure A-12. Case Study 4: Total delay in signal system.

delay should be less than 4.0 vehicle-hours. Total delay was computed using the movement volumes and delays reported in Tables A-35 and A-37, respectively. For the Base alternative, the movement delays at the signalized intersection of Main Street and Sunset Boulevard were worse than LOS D and in excess of 4.0 vehicle-hours. Hence, this intersection's operation was considered "unacceptable."

The Signal alternative was found to satisfy most of the "effectiveness" criteria, as defined in Chapter 3 of the *ESG.* It improved overall system operation and provided acceptable LOS and acceptable operation for both signalized intersections. However, as noted in Table A-38, it increased delay at the subject intersection. As a result, the Signal alternative was not found to be effective.

The Bay alternative was the only alternative found to be effective. This alternative involves re-timing the existing signals at Rio Road and Sunset Boulevard to reflect current traffic volumes and adding a right-turn bay on the eastbound approach at Sunset Boulevard. The new intersection at Elm Street would have two-way stop control.

STAGE 3: ALTERNATIVE SELECTION

During the alternative selection stage, the alternatives advanced from the engineering study stage are reviewed for their potential impact on traffic operations, safety, and the environment. The alternative selection stage consists of the following three steps:

1. Identify impacts,
2. Select best alternative, and
3. Document study.

TABLE A-39 Case Study 4: Alternative effectiveness

Criteria	Alternative		
	Base	Bay	Signal
A. Reduces average system delay?	n.a.	Yes	Yes
B. Acceptable level of service at each signalized intersection?	Yes	Yes	Yes
C. Acceptable operation of minor movements at each signalized intersection?	No	Yes	Yes
D.1 Reduces average intersection delay at subject intersection (Elm Street)?	n.a.	Yes	No
D.2 Acceptable level of service at subject intersection?	Yes	Yes	Yes
D.3 Acceptable operation of minor movements at subject intersection?	Yes	Yes	Yes
Effective alternative?	**No**	**Yes**	**No**

Note: n.a. - not applicable.

Stage 3: Step 1. Identify Impacts

In general, factors are typically identified that might influence alternative selection. These decision factors include: traffic operations, traffic safety, direct cost, environment, and aesthetics. The factors believed to be most relevant included traffic operations, safety, and cost. Impacts to the environment and aesthetics were considered, but reasoned to be unaffected by the proposed alternative.

Stage 3: Step 2. Select Best Alternative

An evaluation of the impacts associated with the Bay alternative indicated that it had relatively low overall impact.

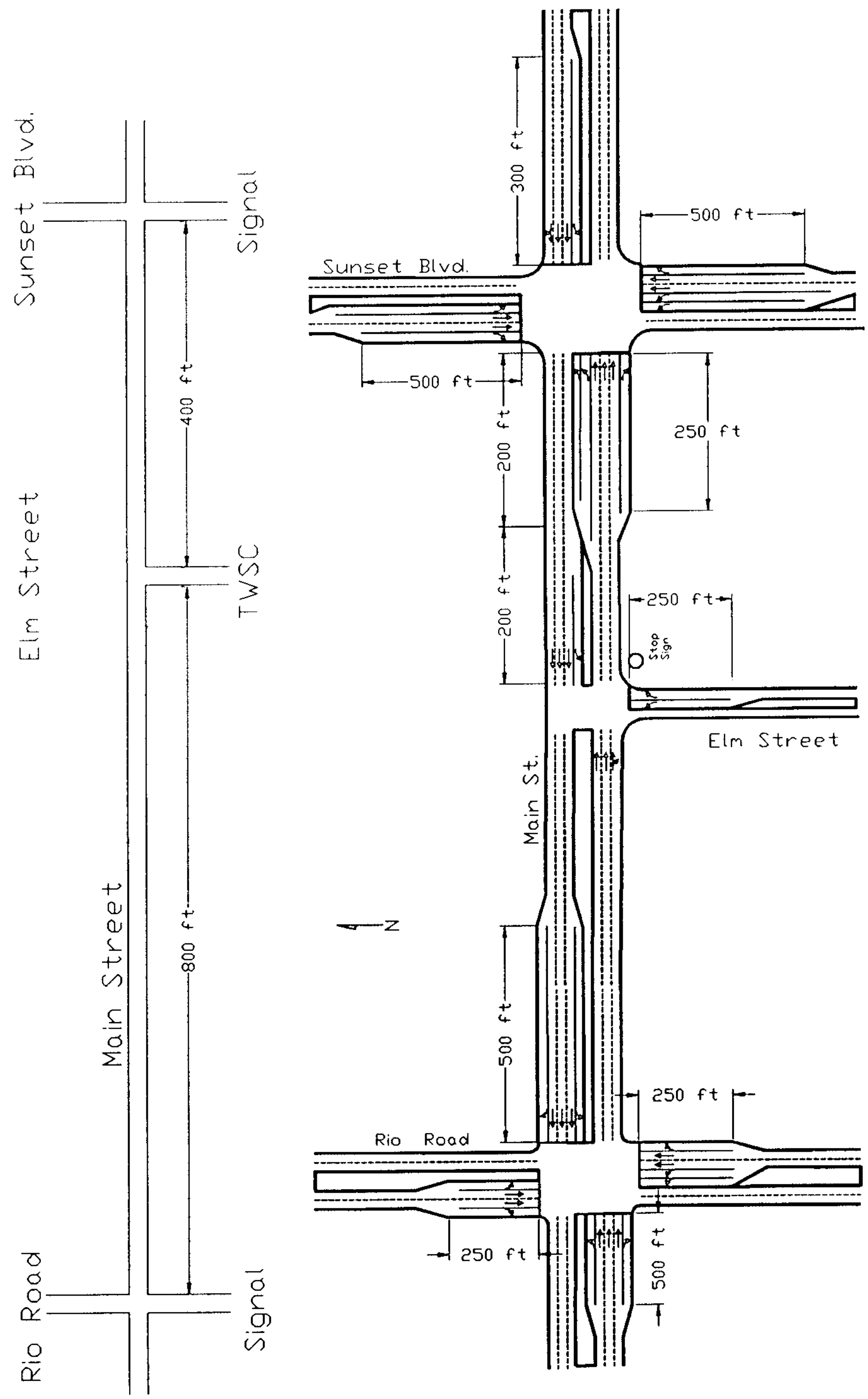


Figure A-13. Case Study 4: Recommended alternative (not to scale).

Based on this analysis, the Bay alternative was selected for implementation. This alternative included construction of a two-way stop-controlled intersection on Main Street, addition of a right-turn bay on the eastbound approach of the intersection at Sunset Boulevard, and adjustment of the arterial signal timing plan to improve traffic progression. A sketch of the recommended alternative is shown in Figure A-13.

Stage 3: Step 3. Document Study

A report documenting the results of the engineering study was prepared and submitted.

REFERENCES

A-1. *Manual of Transportation Engineering Studies.* D.H. Robertson, ed. Prentice-Hall, Englewood Cliffs, New Jersey (1995).

A-2. *Manual on Uniform Traffic Control Devices-Millennium Edition*. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. (December 2000).

A-3. *Highway Capacity Manual 2000*. 4th ed. Transportation Research Board, National Research Council, Washington, D.C. (2000).

A-4. Robinson, B. et al. "Roundabouts: An Informational Guide." *Report No. FHWA-RD-00-067*. Federal Highway Administration, Washington, D.C. (2000).

APPENDIX B

MUTCD TRAFFIC CONTROL SIGNAL WARRANT WORKSHEETS

INTRODUCTION

This appendix describes a series of worksheets that can be used to document the findings of the *MUTCD 2000* traffic control signal warrant check. The following pages illustrate the use of the worksheets for Case Study 1 described in Appendix A. Blank worksheets are provided following this example application.

EXAMPLE APPLICATION

The intersection of County Routes 21 and 27 is in a rural community with approximately 11,000 residents. The intersection has three approach legs; one leg is stop-controlled. County Route 27 (CR 27) is the minor road; it is oriented in an east-west direction and has stop control at the intersection. County Route 21 (CR 21) is the major road; it is oriented in the north-south direction. The speed limit is posted at 45 mph on both roads. All approaches have one traffic lane. Figure B-1 illustrates the intersection geometry.

Crash data were obtained from the state's Department of Public Safety. These data indicated that a total of five collisions occurred at the intersection in the last 12 months (two susceptible to correction by a traffic signal). Regional traffic counts recorded 1 year earlier indicated that the annual average daily traffic demand (AADT) is about 13,500 vehicles per day on CR 21 and 3,000 vehicles per day on CR 27.

Because it was unclear when the 8 highest traffic hours occurred, turn movement volumes were recorded for a total of 13 hours. These counts were adjusted to represent average-day volumes using the techniques described in Appendix C. The adjusted volumes are summarized in Table B-1.

The morning peak demand occurred between 7 and 8 a.m., the afternoon peak occurred between 4 and 5 p.m., and the off-peak hour occurred between 9 and 10 a.m. No pedestrians were observed during the study. Heavy vehicles constituted about 1.0 percent of the traffic stream on each approach. A spot speed study was performed for CR 21; the 85th percentile speed was found to be 50 mph.

A stopped-delay study was performed during the highest-volume hour (i.e., 4 to 5 p.m.). This study focused on the delay to minor-road vehicles. The average delay during the peak hour was 40 s/veh and the total delay was 2.3 veh-h.

A complete warrant analysis for this intersection is summarized in the worksheets on the following pages. The analysis revealed that Warrants 1, 2, and 3 are satisfied and that a traffic control signal may be a viable alternative for the subject intersection.

WORKSHEETS

Blank worksheets for documenting the signal warrant check follow the pages illustrating the use of the worksheets for Case Study 1 described in Appendix A.

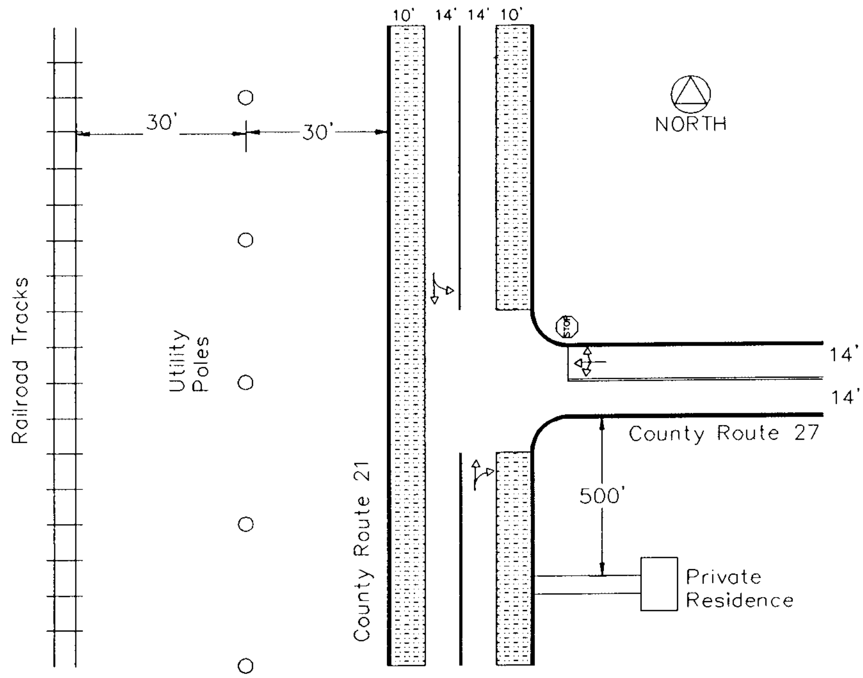


Figure B-1. Existing intersection geometry (not to scale).

TABLE B-1 Turn movement volumes

Hour	Volume (veh)							
	Northbound		Southbound		Major Total	Westbound		Minor Total
	Through	Right	Left	Through		Left	Right	
6 - 7 a.m.	49	36	20	248	353	35	1	36
7 - 8 a.m.	117	109	236	378	840	155	54	209
8 - 9 a.m.	117	109	78	332	636	73	21	94
9 - 10 a.m.	99	109	87	253	548	94	34	128
10 - 11 a.m.	92	91	114	260	557	132	40	172
11 a.m. - 12 p.m.	111	144	200	283	738	152	46	198
12 - 1 p.m.	181	101	54	459	795	56	13	69
1 - 2 p.m.	116	91	187	352	746	152	49	201
2 - 3 p.m.	114	132	154	359	759	140	37	177
3 - 4 p.m.	138	124	127	433	822	158	43	201
4 - 5 p.m.	145	146	143	464	898	140	65	205
5 - 6 p.m.	176	139	129	456	900	152	48	200
6 - 7 p.m.	165	83	136	434	818	164	45	209

TRAFFIC SIGNAL WARRANTS ANALYSIS FORM

Sheet **1** of 5

Calc: *John Smith* Date: *7/ 29/00*

Div.: *Brant* Co.: *Davis* Rte.: *21* Chk: *Bill Jones, P.E.* Date: *8/ 1/00*

Major Road: *County Road 21* Critical approach speed: *45 mph* Lanes: *1*

Minor Road: *County Road 27* Critical approach speed: *45 mph* Lanes: *1*

Volume Level

1. Critical speed of major road traffic > 70 km/h (40 mph) : ☑ Yes ☐ No
2. In built-up area of isolated community of < 10,000 pop.: ☐ Yes ☑ No

If Question 1 or 2 above is answered "Yes" then use "70%" volume level: ☑ 70% ☐ 100%

WARRANT 1 - Eight-Hour Vehicular Volume Satisfied: ☑ Yes ☐ No

Warrant is satisfied if Condition A or Condition B is "100 % satisfied." Warrant is also satisfied if both Condition A and Condition B are "80% satisfied."

- **Condition A - Minimum Vehicular Volume**

100% Satisfied: ☑ Yes ☐ No

80% Satisfied: ☑ Yes ☐ No

Record hours where condition is met and the corresponding volumes in boxes provided. Condition is 100% satisfied if the minimum volumes are met for eight hours. Condition is 80% satisfied if parenthetical volumes are met for eight hours.

(volumes in veh/h)	Minimum Requirements (80% Shown in Brackets)				Hour							
Approach Lanes:	**1**		**2 or more**		*7 - 8 a.m.*	*11 - 12 pm*	*1 - 2 pm*	*2 - 3 pm*	*3 - 4 pm*	*4 - 5 pm*	*5 - 6 pm*	*6 - 7 pm*
Volume Level:	**100%**	**70%**	**100%**	**70%**								
Both Approaches Major Road	500 (400)	350	600 (480)	420	*840*	*738*	*746*	*759*	*822*	*898*	*900*	*818*
Highest Approach Minor Road	150 (120)	105	200 (160)	140	*209*	*198*	*201*	*177*	*201*	*205*	*200*	*209*

- **Condition B - Interruption of Continuous Traffic**

100% Satisfied: ☑ Yes ☐ No

80% Satisfied: ☑ Yes ☐ No

Record hours where condition is met and the corresponding volumes in boxes provided. Condition is 100% satisfied if the minimum volumes are met for eight hours. Condition is 80% satisfied if parenthetical volumes are met for eight hours.

(volumes in veh/h)	Minimum Requirements (80% Shown in Brackets)				Hour							
Approach Lanes:	**1**		**2 or more**		*7 - 8 a.m.*	*11 - 12 pm*	*1 - 2 pm*	*2 - 3 pm*	*3 - 4 pm*	*4 - 5 pm*	*5 - 6 pm*	*6 - 7 pm*
Volume Level:	**100%**	**70%**	**100%**	**70%**								
Both Approaches Major Road	750 (600)	525	900 (720)	630	*840*	*738*	*746*	*759*	*822*	*898*	*900*	*818*
Highest Approach Minor Road	75 (60)	53	100 (80)	70	*209*	*198*	*201*	*177*	*201*	*205*	*200*	*209*

TRAFFIC SIGNAL WARRANTS ANALYSIS FORM

Sheet **2** of 5

WARRANT 2 - Four-Hour Vehicular Volume Satisfied: ☑ Yes ☐ No

Plot four volume combinations on the applicable figure below. If four points lie above the appropriate line, then the warrant is satisfied.

Figure A. Criteria for "100%" volume level.

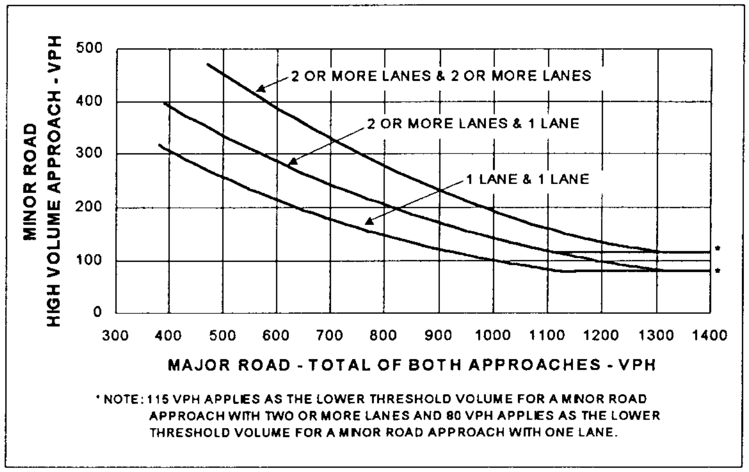
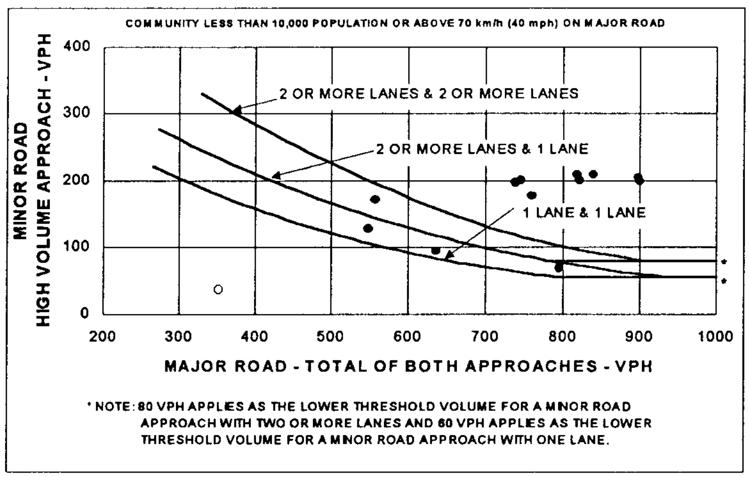


Figure B. Criteria for "70%" volume level.



TRAFFIC SIGNAL WARRANTS ANALYSIS FORM

Sheet **3** of 5

WARRANT 3 - Peak Hour Satisfied: ☑ Yes ☐ No

Unusual condition justifying use of warrant: *High volume from nearby manufacturing plant.*

Record hour where criteria are fulfilled and the corresponding delay or volume in boxes provided. Plot the peak hour volume combination on the applicable figure below. If all three criteria are fulfilled or the plotted point lies above the appropriate line, then the warrant is satisfied.

Criteria	Approach Lanes		No. of Approaches		Hour	Fulfilled?	
	1	2	3	4	*4 - 5 pm*	Yes	No
1. Delay on Minor Approach (veh-h)	4	5			*2.3*		✔
2. Volume on Minor Approach (veh/h)	100	150			*205*	✔	
3. Total Entering Volume (veh/h)			650	800	*1103*	✔	

Figure A. Criteria for "100%" volume level.

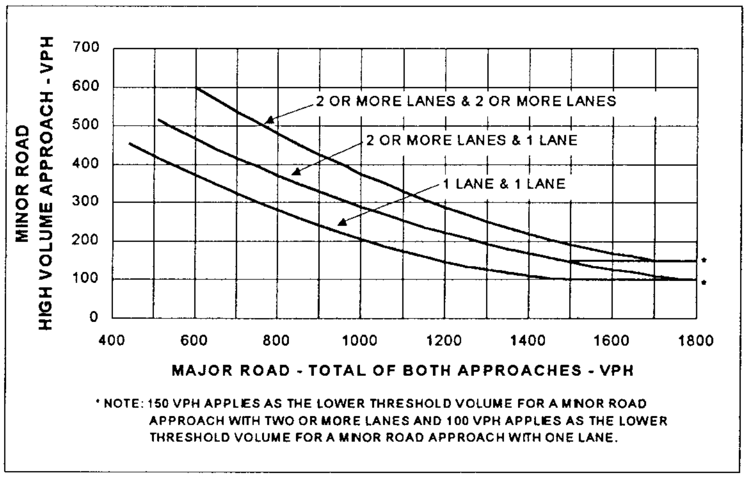
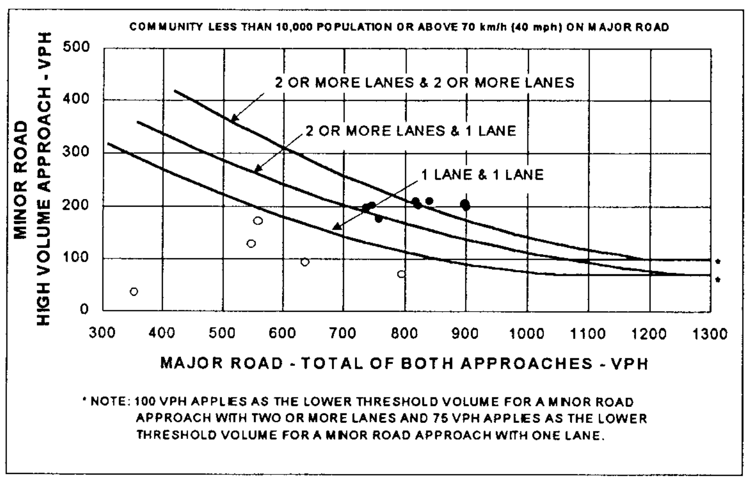


Figure B. Criteria for "70%" volume level.



TRAFFIC SIGNAL WARRANTS ANALYSIS FORM

Sheet **4** of 5

WARRANT 4 - Pedestrian Volume

Satisfied: ☐ Yes ☑ No

Record hours where criteria are fulfilled and the corresponding volume or gap frequency in the boxes provided. The warrant is satisfied if all three of the criteria are fulfilled.

Criteria	Hour				Fulfilled?	
	3-4	4-5	5-6	6-7	Yes	No
1. Pedestrian volume crossing the major road is 100 ped/h or more for each of any four hours or is 190 ped/h or more during any one hour.	0	0	0	0		✔
2. There are less than 60 gaps per hour in the major road traffic stream of adequate length for pedestrians to cross during ths same hours as the pedestrian volume criterion is satisfied.						
3. The nearest traffic signal along the major road is located more than 90 m (300 ft) away. Or, the nearest traffic signal is within 90 m (300 ft) but the proposed traffic signal will not restrict the progressive movement of traffic.						

WARRANT 5 - School Crossing

Applicable: ☐ Yes ☑ No

Satisfied: ☐ Yes ☐ No

Record hour where criteria are fulfilled and the corresponding volume or gap frequency in the boxes provided. The warrant is satisfied if all three of the criteria are fulfilled.

Criteria	Hour	Fulfilled?	
		Yes	No
1. There are a minimum of 20 students during the highest crossing hour.			
2. There are fewer adequate gaps in the major road traffic stream during the period when the children are using the crossing than the number minutes in the same period.			
3. The nearest traffic signal along the major road is located more than 90 m (300 ft) away. Or, the nearest traffic signal is within 90 m (300 ft) but the proposed traffic signal will not restrict the progressive movement of traffic.			

WARRANT 6 - Coordinated Signal System

Satisfied: ☐ Yes ☑ No

Indicate if the criteria are fulfilled in the boxes provided. The warrant is satisfied if either criterion is fulfilled. This warrant should not be applied when the resulting signal spacing would be less than 300 m (1000 ft).

Criteria	Fulfilled?	
	Yes	No
1. On a one-way road or a road that has traffic predominantly in one direction, the adjacent signals are so far apart that they do not provide the necessary degree of vehicle platooning.		✔
2. On a two-way road, adjacent signals do not provide the necessary degree of platooning and the proposed, adjacent signals will collectively provide a progressive operation.		✔

TRAFFIC SIGNAL WARRANTS ANALYSIS FORM

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WARRANT 7 - Crash Experience Satisfied: ☐ Yes ☑ No

Record hours where criteria are fulfilled, the corresponding volume, and other information in the boxes provided. The warrant is satisfied if all three of the criteria are fulfilled.

Criteria		Hour 3 - 4	Hour 4 - 5	Hour 5 - 6	Hour 6 - 7	Met? Yes	Met? No	Fulfilled? Yes	Fulfilled? No
1. One of the warrants to the right is met.	Warrant 4.1 at 80% of volume requirements: 80 ped/h for 4 hrs or 152 ped/h for 1 hr	*0*	*0*	*0*	*0*		✔	✔	
	Warrant 1, Condition A (80% satisfied)					✔			
	Warrant 1, Condition B (80% satisfied)					✔			
2. Adequate trial of other remedial measures has failed to reduce crash frequency.		Measures tried: none							✔
3. Five or more reported crashes, of types susceptible to correction by signal, have occurred within a 12-mo. period.		Number of crashes per 12 months: 2							✔

WARRANT 8 - Roadway Network Satisfied: ☐ Yes ☑ No

Record hours where criteria are fulfilled, the corresponding volume, and other information in the boxes provided. The warrant is satisfied if at least one of the criteria is fulfilled and if all intersecting routes have one or more of the characteristics listed.

Criteria								Met? Yes	Met? No	Fulfilled? Yes	Fulfilled? No
1. Both of the criteria to the right are met.	a. Total entering volume of at least 1,000 veh/h during typical weekday peak hour.	Entering volume: *1103*						✔		✔	
	b. Five-year projected volumes that satisfy one or more of Warrants 1, 2, or 3.	Warrant(s) satisfied: *1, 2, & 3*						✔			
2. Total entering volume at least 1,000 veh/h for each of any 5 hrs of a non-normal business day (Sat. or Sun.)								- Hour			
								- Volume			

Characteristics of Major Routes		Met? Yes	Met? No	Fulfilled? Yes	Fulfilled? No
1. Part of the road or highway system that serves as the principal roadway network for through traffic flow.	Major Road:	✔			✔
	Minor Road:		✔		
2. Rural or suburban highway outside of, entering, or traversing a city.	Major Road:		✔		
	Minor Road:		✔		
3. Appears as a major route on an official plan.	Major Road:	✔			
	Minor Road:		✔		

CONCLUSIONS Warrants Satisfied: *1, 2, 3* Signal Warranted: ☑ Yes ☐ No

Remarks: ____________________

____

TRAFFIC SIGNAL WARRANTS ANALYSIS FORM

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Calc: ____________________ Date: ________

Div.: __________ Co.: __________ Rte.: _____ Chk: ____________________ Date: ________

Major Road: ______________________________ Critical approach speed: __________ Lanes: ________

Minor Road: ______________________________ Critical approach speed: __________ Lanes: ________

Volume Level

1. Critical speed of major road traffic > 70 km/h (40 mph) : ☐ Yes ☐ No
2. In built-up area of isolated community of < 10,000 pop.: ☐ Yes ☐ No

If Question 1 or 2 above is answered "Yes" then use "70%" volume level: ☐ 70% ☐ 100%

WARRANT 1 - Eight-Hour Vehicular Volume Satisfied: ☐ Yes ☐ No

Warrant is satisfied if Condition A or Condition B is "100 % satisfied." Warrant is also satisfied if both Condition A and Condition B are "80% satisfied."

- **Condition A - Minimum Vehicular Volume** 100% Satisfied: ☐ Yes ☐ No
  80% Satisfied: ☐ Yes ☐ No

Record hours where condition is met and the corresponding volumes in boxes provided. Condition is 100% satisfied if the minimum volumes are met for eight hours. Condition is 80% satisfied if parenthetical volumes are met for eight hours.

(volumes in veh/h)	Minimum Requirements (80% Shown in Brackets)				Hour							
Approach Lanes:	**1**		**2 or more**									
Volume Level:	**100%**	**70%**	**100%**	**70%**								
Both Approaches Major Road	500 (400)	350	600 (480)	420								
Highest Approach Minor Road	150 (120)	105	200 (160)	140								

- **Condition B - Interruption of Continuous Traffic** 100% Satisfied: ☐ Yes ☐ No
  80% Satisfied: ☐ Yes ☐ No

Record hours where condition is met and the corresponding volumes in boxes provided. Condition is 100% satisfied if the minimum volumes are met for eight hours. Condition is 80% satisfied if parenthetical volumes are met for eight hours.

(volumes in veh/h)	Minimum Requirements (80% Shown in Brackets)				Hour							
Approach Lanes:	**1**		**2 or more**									
Volume Level:	**100%**	**70%**	**100%**	**70%**								
Both Approaches Major Road	750 (600)	525	900 (720)	630								
Highest Approach Minor Road	75 (60)	53	100 (80)	70								

TRAFFIC SIGNAL WARRANTS ANALYSIS FORM

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WARRANT 2 - Four-Hour Vehicular Volume Satisfied: ☐ Yes ☐ No

Plot four volume combinations on the applicable figure below. If four points lie above the appropriate line, then the warrant is satisfied.

Figure A. Criteria for "100%" volume level.

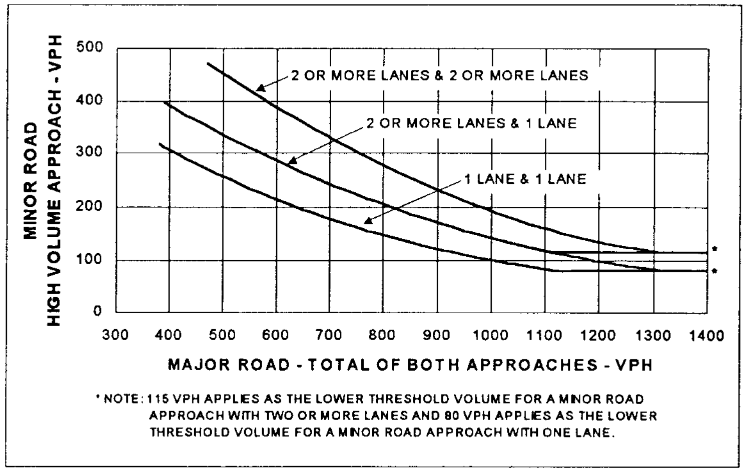
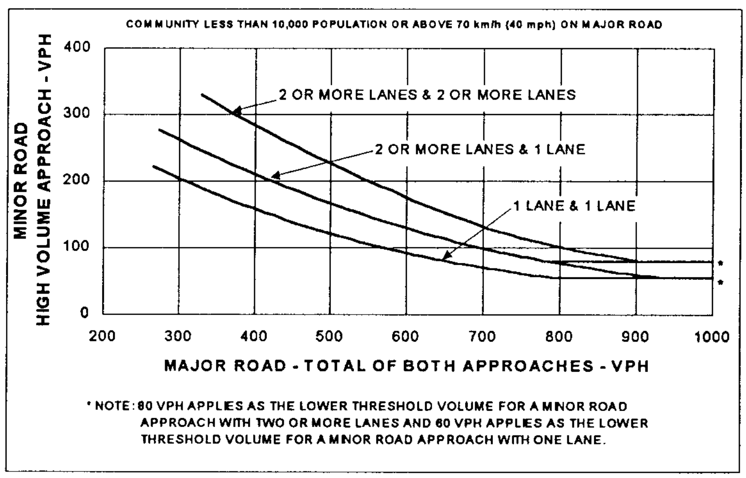


Figure B. Criteria for "70%" volume level.



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WARRANT 3 - Peak Hour Satisfied: ☐ Yes ☐ No

Unusual condition justifying use of warrant: ______________________________

Record hour where criteria are fulfilled and the corresponding delay or volume in boxes provided. Plot the peak hour volume combination on the applicable figure below. If all three criteria are fulfilled or the plotted point lies above the appropriate line, then the warrant is satisfied.

Criteria	Approach Lanes		No. of Approaches		Hour	Fulfilled?	
	1	2	3	4		Yes	No
1. Delay on Minor Approach (veh-h)	4	5					
2. Volume on Minor Approach (veh/h)	100	150					
3. Total Entering Volume (veh/h)			650	800			

Figure A. Criteria for "100%" volume level.

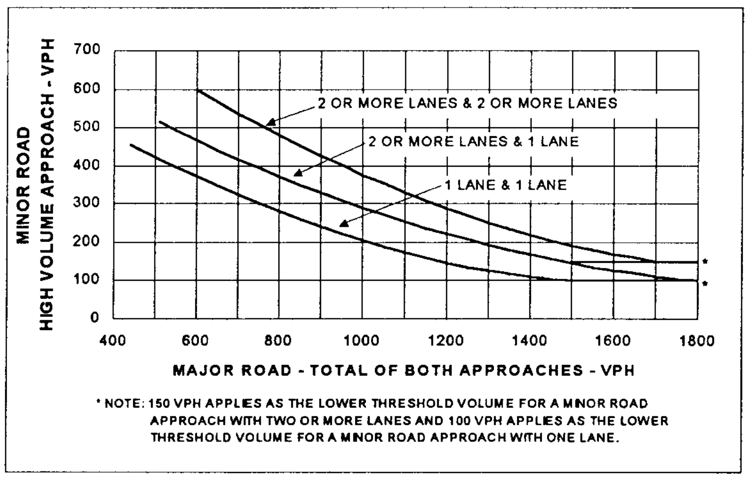
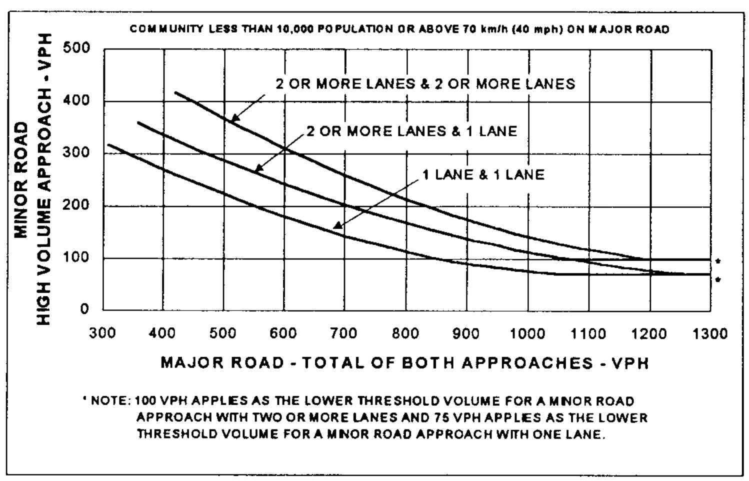


Figure B. Criteria for "70%" volume level.



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WARRANT 4 - Pedestrian Volume Satisfied: ☐ Yes ☐ No

Record hours where criteria are fulfilled and the corresponding volume or gap frequency in the boxes provided. The warrant is satisfied if all three of the criteria are fulfilled.

Criteria	Hour				Fulfilled?	
					Yes	No
1. Pedestrian volume crossing the major road is 100 ped/h or more for each of any four hours or is 190 ped/h or more during any one hour.						
2. There are less than 60 gaps per hour in the major road traffic stream of adequate length for pedestrians to cross during ths same hours as the pedestrian volume criterion is satisfied.						
3. The nearest traffic signal along the major road is located more than 90 m (300 ft) away. Or, the nearest traffic signal is within 90 m (300 ft) but the proposed traffic signal will not restrict the progressive movement of traffic.						

WARRANT 5 - School Crossing Applicable: ☐ Yes ☐ No

Satisfied: ☐ Yes ☐ No

Record hour where criteria are fulfilled and the corresponding volume or gap frequency in the boxes provided. The warrant is satisfied if all three of the criteria are fulfilled.

Criteria	Hour	Fulfilled?	
		Yes	No
1. There are a minimum of 20 students during the highest crossing hour.			
2. There are fewer adequate gaps in the major road traffic stream during the period when the children are using the crossing than the number minutes in the same period.			
3. The nearest traffic signal along the major road is located more than 90 m (300 ft) away. Or, the nearest traffic signal is within 90 m (300 ft) but the proposed traffic signal will not restrict the progressive movement of traffic.			

WARRANT 6 - Coordinated Signal System Satisfied: ☐ Yes ☐ No

Indicate if the criteria are fulfilled in the boxes provided. The warrant is satisfied if either criterion is fulfilled. This warrant should not be applied when the resulting signal spacing would be less than 300 m (1000 ft).

Criteria	Fulfilled?	
	Yes	No
1. On a one-way road or a road that has traffic predominantly in one direction, the adjacent signals are so far apart that they do not provide the necessary degree of vehicle platooning.		
2. On a two-way road, adjacent signals do not provide the necessary degree of platooning and the proposed, adjacent signals will collectively provide a progressive operation.		

TRAFFIC SIGNAL WARRANTS ANALYSIS FORM

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WARRANT 7 - Crash Experience Satisfied: ☐ Yes ☐ No

Record hours where criteria are fulfilled, the corresponding volume, and other information in the boxes provided. The warrant is satisfied if all three of the criteria are fulfilled.

Criteria		Hour				Met? Yes	Met? No	Fulfilled? Yes	Fulfilled? No
1. One of the warrants to the right is met.	Warrant 4.1 at 80% of volume requirements: 80 ped/h for 4 hrs or 152 ped/h for 1 hr								
	Warrant 1, Condition A (80% satisfied)								
	Warrant 1, Condition B (80% satisfied)								
2. Adequate trial of other remedial measures has failed to reduce crash frequency.		Measures tried:							
3. Five or more reported crashes, of types susceptible to correction by signal, have occurred within a 12-mo. period.		Number of crashes per 12 months:							

WARRANT 8 - Roadway Network Satisfied: ☐ Yes ☐ No

Record hours where criteria are fulfilled, the corresponding volume, and other information in the boxes provided. The warrant is satisfied if at least one of the criteria is fulfilled and if all intersecting routes have one or more of the characteristics listed.

Criteria							Met? Yes	Met? No	Fulfilled? Yes	Fulfilled? No
1. Both of the criteria to the right are met.	a. Total entering volume of at least 1,000 veh/h during typical weekday peak hour.	Entering volume:								
	b. Five-year projected volumes that satisfy one or more of Warrants 1, 2, or 3.	Warrant(s) satisfied:								
2. Total entering volume at least 1,000 veh/h for each of any 5 hrs of a non-normal business day (Sat. or Sun.)							- Hour			
							- Volume			

Characteristics of Major Routes		Met? Yes	Met? No	Fulfilled? Yes	Fulfilled? No
1. Part of the road or highway system that serves as the principal roadway network for through traffic flow.	**Major Road:**				
	Minor Road:				
2. Rural or suburban highway outside of, entering, or traversing a city.	**Major Road:**				
	Minor Road:				
3. Appears as a major route on an official plan.	**Major Road:**				
	Minor Road:				

CONCLUSIONS Warrants Satisfied:__________ Signal Warranted: ☐ Yes ☐ No

Remarks: ______________________________________

APPENDIX C

DATA FOR GUIDELINE EVALUATION

INTRODUCTION

This appendix describes the data collection requirements for the engineering assessment process. These requirements are categorized into the following sections:

1. Procedures for Collecting Site Data and
2. Procedures for Estimating Traffic Data.

The first section describes procedures for collecting data needed for the alternative identification and screening stage of the assessment process. The second section describes a procedure for estimating the data needed for the engineering study stage. Blank worksheets for recording the data are included in the third section of this appendix.

PROCEDURES FOR COLLECTING SITE DATA

This section describes two data collection activities for the alternative identification and screening stage of the assessment process. These data are needed to define the operational problems at the subject intersection and identify their causes. One data collection activity includes a site visit where a condition diagram and an on site observation report are completed. A second data collection activity requires the acquisition of crash records for the site and a presentation of the crash history in a collision diagram. The activities associated with the development of these summary documents are described in the next three sections.

Condition Diagram

The condition diagram represents a complete physical record of relevant site characteristics. The diagram consists of a plan-view, scale drawing of the features in the vicinity of the subject intersection. Some of the items that might be shown on the diagram include

- Names of major and minor road;
- Functional class (arterial, collector, local);
- Intersection angle;
- Lane configuration and lane use markings;
- Approach grade;
- Lane widths;
- Length of turn bays;
- Location and legend of traffic signs;
- Location of on-street parking;
- Location of medians and islands;
- Location of transit stops;
- Location of driveways;
- Location of any roadway lights;
- Surrounding land uses;
- Location of sidewalks and crosswalks;
- Location of drainage structures;
- Distance to nearest traffic signals;
- Nearby railroad crossings; and
- Location of potential sight obstructions (may include vegetation, utility poles, fences, buildings, mailboxes, and controller cabinets).

The items shown on the diagram can vary, depending on the analyst's assessment of the relevance of various items (as they relate to the reported problems) and the anticipated extent of potential improvements. For example, an anticipated signing change may not require as much detail as when traffic lanes are proposed to be added to an approach. A sample condition diagram is shown in Figure C-1.

An accurate and complete diagram will help the engineer select alternatives that are appropriate for the site. Location measurements should be performed with a distance measuring wheel (or odometer). This device offers a good balance between precision and ease of measurement.

Photographs of the site may also be taken to supplement the information obtained from the sketch. Photos can provide a visual record of the conditions present at the intersection.

A blank condition diagram is included in the Worksheets section of this appendix.

On Site Observation Report

An important component of the problem-cause identification process is the firsthand observation of traffic operations. As such, it is essential that the engineer responsible for the assessment be present during the site visit. This visit should be scheduled to coincide with the occurrence of the reported problems (e.g., p.m. peak traffic demand hour).

The observational study should include several components. First, the engineer should drive through the subject intersection and attempt to experience the problem, especially if it relates to sight distance limitations. Then, the engineer should observe traffic operations at the intersection. An On Site Observation Report, as shown in Figure C-2, should be completed during the site visit. A blank report form is included in the Worksheets section of this appendix.

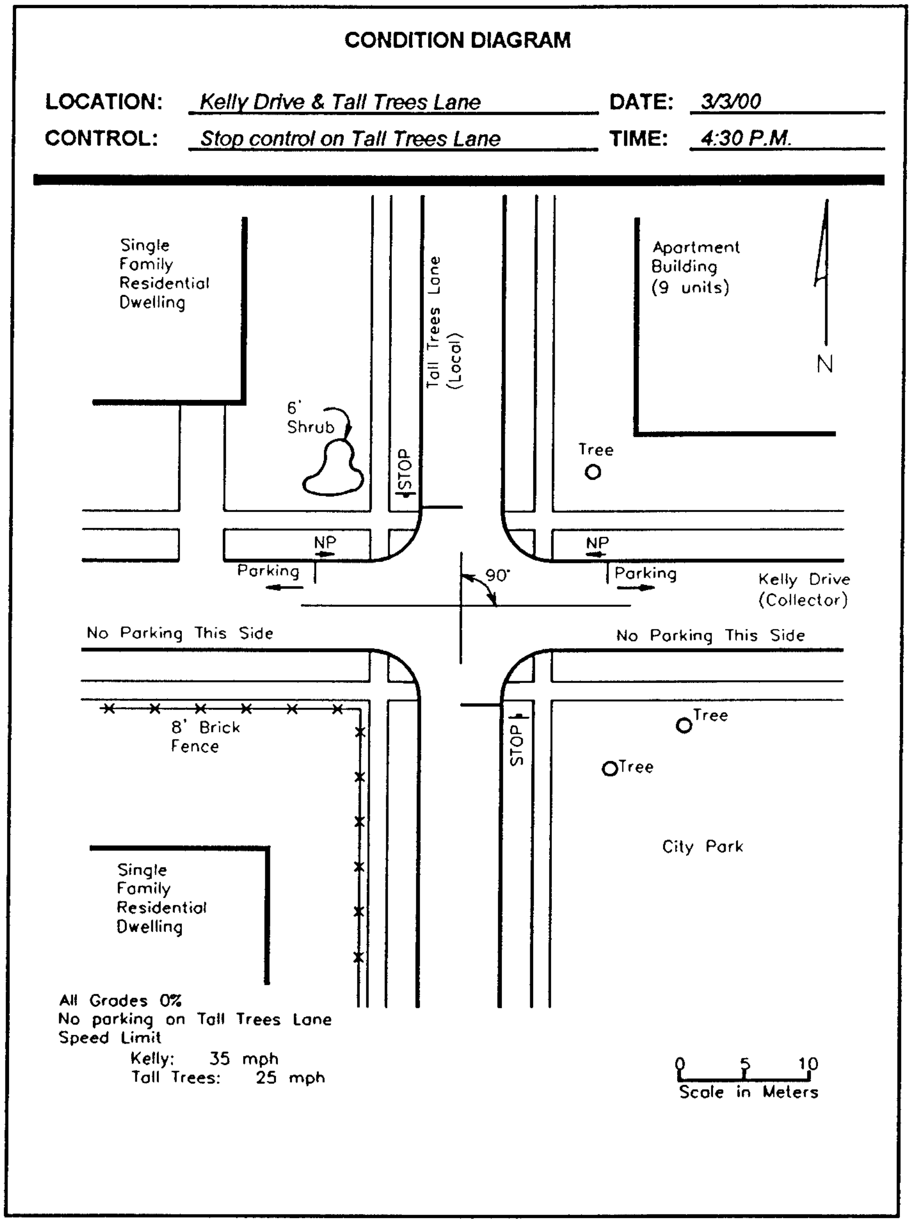


Figure C-1. Sample condition diagram.

During the observational study, the engineer should attempt to identify the circumstances that contribute to the reported (or observed) problems. The questions included in the report are intended to direct the engineer's attention to situations that often cause problems at intersections. However, the engineer should also look for other site-specific factors that may contribute to the problem.

The questions on the report form are answered by checking "No," "Not Sure," or "Yes." A response of "Not Sure" or "Yes" should be considered as an indication that a problem may exist at the intersection. The engineer should describe the problem and its likely cause(s) in the Comments section of the report.

Collision Diagram

The collision diagram documents the spatial orientation of the crashes that have occurred at the intersection. The diagram shows all crashes that have occurred during the past 1 to 3 years. The collision diagram includes a sketch (without scale) of the intersection curb lines and identifies roads by their name and direction. A symbol drawing is then added to the diagram for each crash that occurred. The basic characteristics of each accident, such as date, day of week, time of crash, weather, pavement condition, and number of injuries, are recorded next to each symbol. A sample collision diagram is shown in Figure C-3.

ON SITE OBSERVATION REPORT

LOCATION: *Kelly Drive & Tall Trees Lane* **DATE:** *3/3/00*

CONTROL: *Stop control on Tall Trees Lane* **TIME:** *4:30 P.M.*

Isolated and Non-Isolated Intersections	**No**	**Not Sure**	**Yes**
1. Does road curvature, vegetation, buildings, parked cars, etc. block drivers' views of conflicting vehicles?	✓		
2. Is the intersection skew angle so sharp that it makes it difficult to view conflicting vehicles or complete turns?	✓		
3. Do vehicle speeds appear too high?		✓	
4. Does the delay for the minor-road right-turn appear excessive?		✓	
5. Does the delay for the minor-road through appear excessive?	✓		
6. Does the delay for the minor-road left-turn appear excessive?			✓
7. Does the delay for the major-road left-turn appear excessive	✓		
8. Does the queue for the major-road left-turn ever impede major-road through traffic?	✓		
9. As major-road vehicles slow to turn, do they impede other vehicles?			✓
10. Do parking maneuvers impede other vehicles?	✓		
11. Are drivers not complying with the traffic control devices?	✓		
12. Is there evidence that one or more curb radii are too small?	✓		
13. Do pedestrians appear to cause conflict with vehicular traffic?	✓		
14. Are there guidance or control problems that could be mitigated by raised-curb channelization?	✓		
Non-Isolated Intersections			
A. Do queues from adjacent signalized intersections spillback into the subject intersection?	*na*		
B. Do vehicles slowing to turn at adjacent intersections or driveways contribute to the delay to major- or minor-road drivers?	*na*		
C. Is it possible that some drivers are diverting to the subject intersection because of congestion on a nearby arterial street?	*na*		
D. Does the arrival pattern of major-road traffic platoons contribute to the delay to minor-road drivers?	*na*		

na = not applicable.

Comments:

Figure C-2. Sample onsite observation report.

The information provided on the diagram can provide important clues to the cause of the reported problem. For example, frequent right-angle collisions may indicate that sight lines are obstructed; frequent nighttime collisions may indicate a need for roadway lighting. More information on the preparation of and interpretation of collision diagrams is provided by Hummer (*C-1*). A blank collision diagram worksheet is provided in the Worksheets section of this appendix.

PROCEDURES FOR ESTIMATING TRAFFIC DATA

This section describes a procedure for estimating the approach and turning movement volumes at a proposed intersection. The objective of the procedure is to estimate these volumes for the average day. The "average day" is defined as a day representing traffic volumes normally and repeatedly found at a location. The average day is typically a weekday when volumes are influenced by employment; however, it could be a weekend day when volumes are influenced by entertainment or recreation.

The procedure is based on the assumption that the intersection does not exist at the time of the study and that only an estimate of the average daily traffic (ADT) is available for each of the intersecting roadways. If one of the roads exists and a traffic count is taken on it, steps are described for adjusting the count to offset seasonal variations in traffic demand. If hourly

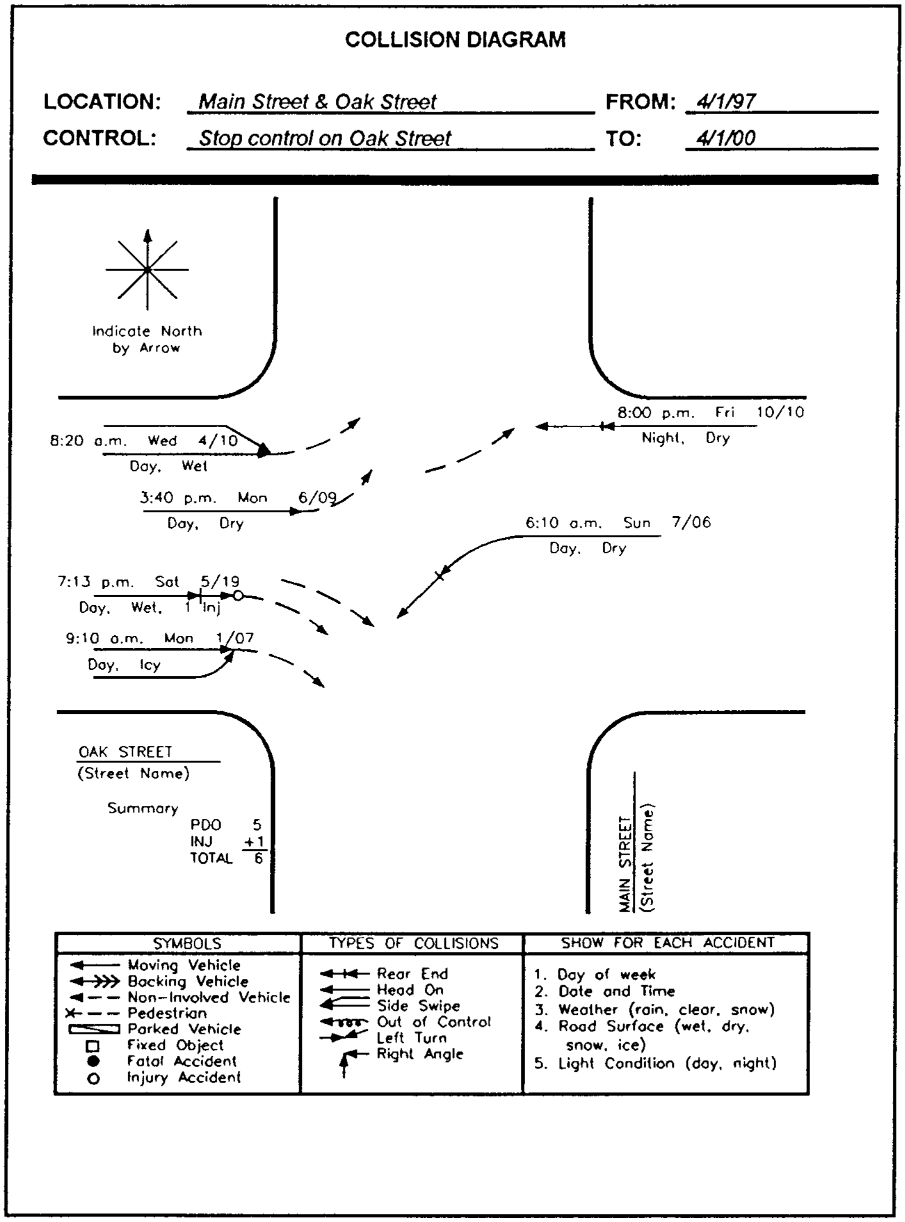


Figure C-3. Sample collision diagram.

traffic volumes on each intersection approach are available from existing counts or from a traffic assignment process associated with the proposed intersection, then Steps 1, 2, and 3 can be skipped.

Step 1. Estimate Volume Distribution Factors and Turn Percentages

As a first step, the volume distribution factors and turn movement percentages are estimated. The volume distribution factor D represents the percentage of the two-way volume on a given intersection leg at the intersection. Typical percentages range from 40 to 60 percent. If local field measurements are not available, then D can be estimated using the factors in Table C-1.

The turn movement percentages must also be established in this step. The percentages in Table C-1 can be used if estimates based on local traffic patterns are unavailable. These percentages were reported by Hauer et al. (*C-2*) based on an analysis of turn volumes on 283 intersection approaches.

Step 2. Obtain Volume Adjustment Factors

The procedure requires adjustment factors that reflect hourly, weekly, and monthly volume variations for roadways in the vicinity of the proposed intersection. These factors are

TABLE C-1 Typical volume distribution factors and turn movement percentages

Volume Distribution Factor (percent)			Turn Movement Percentages		
Travel Direction	Evaluation Period		Facility Type	Movement	
	AM or PM Peak	Off-Peak		Left-Turn	Right-Turn
Inbound	60	50	Central business district	10	12
Outbound	40	50	Arterial to arterial	12	12
			Arterial to collector	4	5
			Collector to arterial	30	32
			Collector to collector	10	20

typically obtained from the permanent network of control count locations established by the state's Department of Transportation. Table C-2 illustrates these factors for a typical urban location. These factors are for illustrative purposes only; the analyst must obtain similar factors from the local state agency.

Step 3. Compute Approach Volumes

The approach volumes for a given hour are estimated using the hourly adjustment factors from Table C-2 and the volume distribution factors identified in Step 1. The equation used for this computation is:

$$V_{a,i} = ADT \times \frac{f_{h,i}}{100} \times D_i \qquad \text{(C-1)}$$

where,

$V_{a,i}$ = approach volume during hour i ($i = 1, 2, 3, \ldots, 24$), veh/h;
ADT = average daily traffic, veh/d;
$f_{h,i}$ = hourly volume adjustment factor for hour i (percent of AADT); and
D_i = volume distribution factor for hour i.

The *ADT* variable can represent the annual average daily traffic (AADT) or an estimate of it based on the planning process. It can also represent a 24-hour count made on a

TABLE C-2 Illustrative hourly, weekly, and monthly volume adjustment factors

Hour of Day	Percent of AADT	Day of Week	Percent of Average
Midnight - 1 a.m.	1.2	Sunday	77
1 a.m. - 2 a.m.	0.8	Monday	103
2 a.m. - 3 a.m.	0.5	Tuesday	101
3 a.m. - 4 a.m.	0.4	Wednesday	100
4 a.m. - 5 a.m.	0.5	Thursday	106
5 a.m. - 6 a.m.	1.0	Friday	113
6 a.m. - 7 a.m.	2.5	Saturday	102
7 a.m. - 8 a.m.	7.7	**Average weekday:**	**105**
8 a.m. - 9 a.m.	5.6		
9 a.m. - 10 a.m.	5.0	**Month of Year**	**Percent of Average**
10 a.m. - 11 a.m.	5.2	January	95
11 a.m. - Noon	5.5	February	104
Noon - 1 p.m.	6.2	March	87
1 p.m. - 2 p.m.	6.0	April	95
2 p.m. - 3 p.m.	6.1	May	99
3 p.m. - 4 p.m.	6.2	June	104
4 p.m. - 5 p.m.	7.8	July	102
5 p.m. - 6 p.m.	9.3	August	104
6 p.m. - 7 p.m.	6.1	September	100
7 p.m. - 8 p.m.	5.2	October	99
8 p.m. - 9 p.m.	3.8	November	104
9 p.m. - 10 p.m.	3.1	December	105
10 p.m. - 11 p.m.	2.5		
11 p.m. - Midnight	1.8		

specific day, in which case, the day-of-week and month-of-year adjustments described in Step 5 should be used to remove any bias due to weekly and monthly volume variations.

To illustrate the application of Equation C-1, consider a proposed intersection on an existing road. The existing road is a major arterial having an AADT of 10,000 veh/d. The major road is oriented in a north-south direction. The minor road is planned as a collector with an estimated ADT of 5,000 veh/d. The intersection is located on the northeast side of a major city. During the afternoon peak hour, traffic flow is predominantly in the northbound and eastbound directions. The volume distribution is estimated as 60 percent in each of these travel directions (and 40 percent in the opposite directions) during the afternoon peak. This trend is reversed during the morning peak.

The intersection approach volumes calculated using Equation C-1 are shown in Table C-3 for the eight highest hours of the day. Volumes for these hours were computed to facilitate the warrant check. The turn movement volumes shown are the subject of discussion in the next calculation step.

Step 4. Compute Turn Movement Volumes

The turn movement volumes are computed using the turn percentages identified in Step 1. For each approach, the turn movement volume is computed as the product of the turn percentage and the approach volume. Table C-3 illustrates this computation for the eastbound approach. The turn percentages were obtained from Table C-1 for the "Collector to arterial" facility type.

Occasionally, the turn movement volumes obtained from this procedure are not in reasonable proportion to the original ADT volumes associated with each intersection leg. This disparity may occur when one leg ADT is unusually large (or small) relative to the other leg ADTs. If this situation occurs, the procedure described by Hauer et al. (*C-2*) or that provided in Chapter 10 of the *Highway Capacity Manual 2000* (*C-3*) can be used to refine the estimate of turn movement volumes.

Step 5. Compute Average-Day Volumes

Two adjustments to the volume data are considered during this step. First, an adjustment is needed if the volumes used are based on counts taken on a specific day of the week and month of the year. This adjustment converts the count data to average-day-of-year volumes. It can be applied to the count data in whatever form it is collected (e.g., 24-hour two-way total or 6-hour turn movement count); however, this procedure illustrates the adjustment being applied to the estimated turn movement counts. The equation for making this adjustment is:

$$V_{t,doy} = V_t \times \frac{100}{f_w} \times \frac{100}{f_m} \qquad \text{(C-2)}$$

where

$V_{t,doy}$ = turn movement volume reflecting the average-day-of-year, veh/h;
V_t = turn movement volume count, veh/d;
f_w = weekly volume adjustment factor (percent of AADT); and
f_m = monthly volume adjustment factor (percent of AADT).

If the turn movement volumes are derived from AADT values or otherwise represent average-day-of-year volumes, then the adjustment using Equation C-2 is not needed.

The second adjustment converts the average-day-of-year volume to the average-day volume required for the engineer-

TABLE C-3 Example computation of approach volumes

Hour of Day	Percent of AADT	Approach Volume (veh/h)[1]				Eastbound Turn Movement Volume (veh/h)		
		Major Road		Minor Road				
		NB	SB	EB	WB	Left	Through	Right
7 a.m. - 8 a.m.	7.7	308	462	154	231	46	59	49
Noon - 1 p.m.	6.2	310	310	155	155	47	59	50
1 p.m. - 2 p.m.	6.0	300	300	150	150	45	57	48
2 p.m. - 3 p.m.	6.1	305	305	153	153	46	58	49
3 p.m. - 4 p.m.	6.2	310	310	155	155	47	59	50
4 p.m. - 5 p.m.	7.8	468	312	234	156	70	89	75
5 p.m. - 6 p.m.	9.3	558	372	279	186	84	106	89
6 p.m. - 7 p.m.	6.1	305	305	153	153	46	58	49
AM peak hr. vol. dist. factor:		40	60	40	60	**Turn Percentages**		
PM peak hr. vol. dist. factor:		60	40	60	40	30	38	32
Off-peak vol. dist. factor:		50	50	50	50			

Note:
1 - Forecast ADTs: North-South = 10,000 veh/d; East-West = 5,000 veh/d.

TABLE C-4 Example computation of average-day volumes

Hour of Day	Eastbound Turn Movement Vol. (veh/h)[1]			Average-Day Turn Movement Vol. (veh/h)		
	Left	Through	Right	Left	Through	Right
7 a.m. - 8 a.m.	46	59	49	51	65	54
Noon - 1 p.m.	47	59	50	51	65	55
1 p.m. - 2 p.m.	45	57	48	50	63	53
2 p.m. - 3 p.m.	46	58	49	51	64	54
3 p.m. - 4 p.m.	47	59	50	51	65	55
4 p.m. - 5 p.m.	70	89	75	78	98	83
5 p.m. - 6 p.m.	84	106	89	93	117	99
6 p.m. - 7 p.m.	46	58	49	51	64	54

Note:
1 - Count taken on Wednesday (f_w = 100) in April (f_m = 95).

ing study. The average-day volume is computed by adjusting the average-day-of-year volume to reflect the average weekday. The data in Table C-2 suggest that the average weekday volume is 105 percent of the average-day-of-year volume. This percentage is based on the average of the five weekday averages listed in Table C-2. A similar trend may be found in the adjustment factors obtained in Step 2 for the subject location.

The average-day volume is computed using the following equation:

$$V_t{}^* = V_{t,doy} \times \frac{f_{ad}}{100} \qquad (C-3)$$

where

$V_t{}^*$= average-day turn movement volume, veh/h; and
f_{ad} = average weekday adjustment factor.

The use of Equations C-2 and C-3 are illustrated in Table C-4. This example assumes that the ADTs used to estimate the turn movement volumes were obtained from a 24-hour count taken on a Wednesday in April. The factors listed in Table C-2 indicate that Wednesday is about equal to the average day-of-week; however, a count in April may actually underestimate the true annual average daily volume by 5 percent. Therefore, Equation C-2 is needed to convert the turn movement volumes to average-day-of-year volumes. Then, Equation C-3 is needed to convert these volumes to average-day volumes. The total adjustment from both equations combined represents an 11.0-percent (= 1.05/0.95/1.00) increase in the measured volumes. The resulting, average-day volumes are listed in the last three columns of Table C-4.

This step of the procedure was illustrated using the more typical case where the weekdays are a better representation of the "average day" than the weekend days. If the weekend days are a better reflection of the "average day," then the variable f_{ad} would be estimated as the average of the Saturday and Sunday day-of-week volume percentages. The other computations would be the same as previously stated.

WORKSHEETS

Blank worksheets for the following studies and analyses follow the Reference section:

1. Condition Diagram,
2. On Site Observation Report,
3. Collision Diagram,
4. Critical Volume Worksheet, and
5. Controller Setting Worksheet.

REFERENCES

C-1. Hummer, J.E. "Chapter 11 - Traffic Accident Studies." *Manual of Transportation Engineering Studies.* D.H. Robertson, ed. Prentice-Hall, Englewood Cliffs, New Jersey (1995).

C-2. Hauer, E., Pagitsas, E., and Shin, B.T. "Estimation of Turning Flows from Automatic Counts." *Transportation Research Record 795.* Transportation Research Board, National Research Council, Washington, D.C. (1981) pp. 1–7.

C-3. *Highway Capacity Manual 2000.* 4th ed. Transportation Research Board, National Research Council, Washington, D.C. (2000).

CONDITION DIAGRAM

LOCATION: ______________________________ DATE: ______________

CONTROL: ______________________________ TIME: ______________

Indicate North with Arrow

ON SITE OBSERVATION REPORT

LOCATION: ______________________ DATE: ____________

CONTROL: ______________________ TIME: ____________

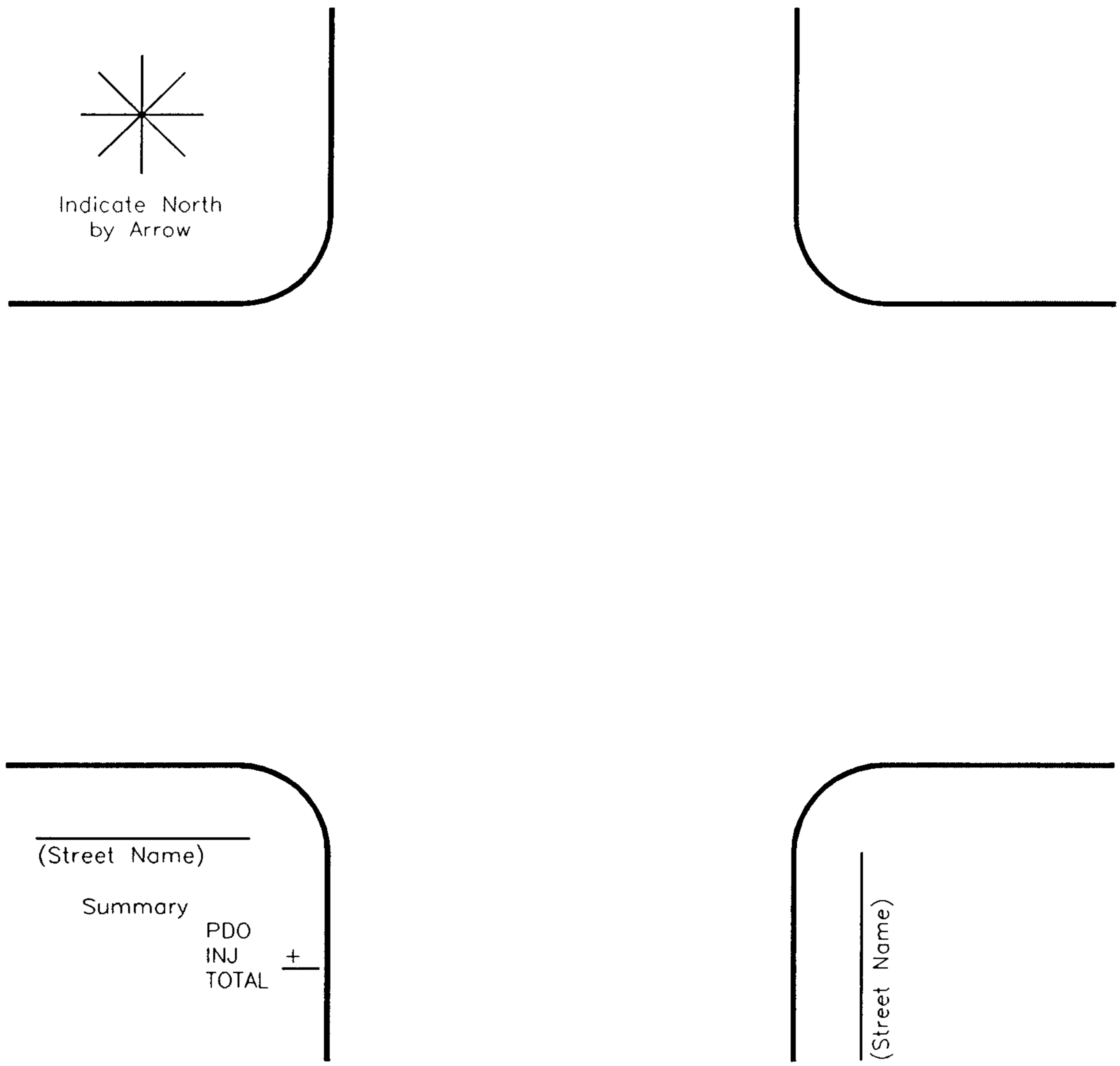
Isolated and Non-Isolated Intersections	No	Not Sure	Yes
1. Does road curvature, vegetation, buildings, parked cars, etc. block drivers' views of conflicting vehicles?	___	___	___
2. Is the intersection skew angle so sharp that it makes it difficult to view conflicting vehicles or complete turns?	___	___	___
3. Do vehicle speeds appear too high?	___	___	___
4. Does the delay for the minor-road right-turn appear excessive?	___	___	___
5. Does the delay for the minor-road through appear excessive?	___	___	___
6. Does the delay for the minor-road left-turn appear excessive?	___	___	___
7. Does the delay for the major-road left-turn appear excessive	___	___	___
8. Does the queue for the major-road left-turn ever impede major-road through traffic?	___	___	___
9. As major-road vehicles slow to turn, do they impede other vehicles?	___	___	___
10. Do parking maneuvers impede other vehicles?	___	___	___
11. Are drivers not complying with the traffic control devices?	___	___	___
12. Is there evidence that one or more curb radii are too small?	___	___	___
13. Do pedestrians appear to cause conflict with vehicular traffic?	___	___	___
14. Are there guidance or control problems that could be mitigated by raised-curb channelization?	___	___	___
Non-Isolated Intersections			
A. Do queues from adjacent signalized intersections spillback into the subject intersection?	___	___	___
B. Do vehicles slowing to turn at adjacent intersections or driveways contribute to the delay to major- or minor-road drivers?	___	___	___
C. Is it possible that some drivers are diverting to the subject intersection because of congestion on a nearby arterial street?	___	___	___
D. Does the arrival pattern of major-road traffic platoons contribute to the delay to minor-road drivers?	___	___	___

Comments:

COLLISION DIAGRAM

LOCATION: ______________________________ **FROM:** ______________

CONTROL: ______________________________ **TO:** ______________



SYMBOLS	TYPES OF COLLISIONS	SHOW FOR EACH ACCIDENT
Moving Vehicle Backing Vehicle Non–Involved Vehicle Pedestrian Parked Vehicle Fixed Object Fatal Accident Injury Accident	Rear End Head On Side Swipe Out of Control Left Turn Right Angle	1. Day of week 2. Date and Time 3. Weather (rain, clear, snow) 4. Road Surface (wet, dry, snow, ice) 5. Light Condition (day, night)

CRITICAL VOLUME WORKSHEET								
General Information								
Location: ______________________________					**Analysis Period:** ______ to ______			
Volume and Lane Geometry Input								
Approach:	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Movement, No.: [1]	LT, 5	TH+RT, 2	LT, 1	TH+RT, 6	LT, 3	TH+RT, 8	LT, 7	TH+RT, 4
Volume (v_i), veh/h i = 1, 2, 3, ... 8								
Lanes (n_i)								
Phase Sequence	**1 Phase** (protected through & permitted left)				**1 Phase** (protected through & permitted left)			
Opposing Volume ($v_{o,i}$), veh/h	v_6 =		v_2 =		v_4 =		v_8 =	
LT equivalence ($E_{L,i}$) (Fig. 3-8)		1.0		1.0		1.0		1.0
Sneakers (S_i), veh/h	90	0.0	90	0.0	90	0.0	90	0.0
Adjusted volume (v^*_i) [= $E_L (v_i - S_i) \geq 0.0$]								
Lane volume ($v_{n,i}$) [= v^*_i / n_i]{see note 2}								
Critical volumes (v_c), veh/h/ln	Larger of: ($v_{n,1}, v_{n,2}, v_{n,5}, v_{n,6}$) =				Larger of: ($v_{n,3}, v_{n,4}, v_{n,7}, v_{n,8}$) =			
	2 Phase (with protected-plus-permitted left)				**2 Phase** (with protected-plus-permitted left)			
Permitted capacity ($c_{p,i}$), veh/h	60	0.0	60	0.0	60	0.0	60	0.0
Adjusted volume (v^*_i) [= $(v_i - c_{p,i}) \geq 0.0$]								
Lane volume ($v_{n,i}$) [= v^*_i / n_i], veh/h/ln								
Critical volumes (v_c) {see note 3}, veh/h/ln	Larger of: ($v_{n,1}, v_{n,5}$)=		Larger of: ($v_{n,2}, v_{n,6}$)=		Larger of: ($v_{n,3}, v_{n,7}$)=		Larger of: ($v_{n,4}, v_{n,8}$)=	
	2 Phase (with protected-only left)				**2 Phase** (with protected-only left)			
Lane volume ($v_{n,i}$) [= v_i / n_i], veh/h/ln								
Critical volumes (v_c) {see note 3}, veh/h/ln	Larger of: ($v_{n,1}, v_{n,5}$)=		Larger of: ($v_{n,2}, v_{n,6}$)=		Larger of: ($v_{n,3}, v_{n,7}$)=		Larger of: ($v_{n,4}, v_{n,8}$)=	

Notes:

1 - Numbers follow NEMA movement-based phase numbering system (LT = left-turn, TH = through, RT = right-turn).

2 - If there is no left-turn lane for a given approach, then the lane volume for the left-turn movement equals 0.0 and the lane volume for the through movement is based on the total, adjusted approach volume. For example, if the eastbound approach has no left-turn lane (i.e., $n_5 = 0$), then $v_{n,5} = 0.0$ and $v_{n,2} = (v^*_5 + v^*_2)/ n_2$.

3 - Critical volume for protected left-turn phases is based on the following assumptions: (1) left-turn phases lead adjacent through phases, (2) one or more exclusive left-turn lanes exist, and (3) both left-turn movements are protected.

CONTROLLER SETTING WORKSHEET								
General Information								
Location: ______					**Analysis Period:** ______ to ______			
Change Interval and Minimum Green								
Approach:	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Movement, No.: [1]	LT, 5	TH+RT,2	LT, 1	TH+RT,6	LT, 3	TH+RT,8	LT, 7	TH+RT,4
Yellow + all-red (Y_i), s (Table 3-6) {see note 2}	$Y_2 =$		$Y_6 =$		$Y_8 =$		$Y_4 =$	
Ped. phase time ($P_{p,i}$), s (Table 3-6) {see note 3}	0.0		0.0		0.0		0.0	
Minimum green ($G_{m,i}$), s [= larger of: ($P_{p,i}$-Y_i, 8.0)]								
Critical Volume Summary {see note 4}								
1 phase E-W (n_p = 2) 1 phase N-S	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) =				Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) =			
1 phase E-W (n_p = 3) 2 phases N-S	Larger of: ($v_{n,1}$, $v_{n,2}$, $v_{n,5}$, $v_{n,6}$) =				Larger of: ($v_{n,3}$,$v_{n,7}$)=		Larger of: ($v_{n,4}$,$v_{n,8}$)=	
2 phases E-W (n_p = 3) 1 phase N-S	Larger of: ($v_{n,1}$,$v_{n,5}$)=		Larger of: ($v_{n,2}$,$v_{n,6}$)=		Larger of: ($v_{n,3}$, $v_{n,4}$, $v_{n,7}$, $v_{n,8}$) =			
2 phases E-W (n_p = 4) 2 phases N-S	Larger of: ($v_{n,1}$,$v_{n,5}$)=		Larger of: ($v_{n,2}$,$v_{n,6}$)=		Larger of: ($v_{n,3}$,$v_{n,7}$)=		Larger of: ($v_{n,4}$,$v_{n,8}$)=	
Sum of critical volumes (Σ v_c), veh/h/ln	Σ v_c = ______ No. phases (n_p) = ______				Min. Delay Cycle (C_o) = ______ (Fig. 3-6) Cycle Length (C) = ______			
Pretimed Phases	Pretimed (or non-actuated) phase time				Pretimed (or non-actuated) phase time			
Critical volumes by phase ($v_{c,i}$), veh/h/ln {see note 5}								
Green duration (G), s [= $v_{c,i}/\Sigma v_c$ (C - 4n_p) + 4 -Y_i]								
Actuated Phases	Actuated phase maximum green setting				Actuated phase maximum green setting			
Lane volume ($v_{n,i}$), veh/h/ln (from Critical Vol. Wksht.)								
Min.-delay green ($G_{o,i}$) s [= $v_{n,i}$/ Σv_c (C_o - 4n_p) + 4 -Y_i]								
Maximum green setting, s [= larger of:($G_{m,i}$+12,1.3$G_{o,i}$)]								
Unit extension, s (Table 3-10)								

Notes:

1 - Numbers follow NEMA movement-based phase numbering system (LT = left-turn, TH = through, RT = right-turn). Complete the columns for the left-turn movements (i = 1, 3, 5, 7) *only* if a corresponding left-turn phase exists.

2 - Compute the change interval (Y + AR) for the through phases only. If a left-turn phase exists then set its change interval equal to that associated with the adjacent through movement.

3 - If pedestrians are served on a through phase that does not have pedestrian detection (i.e., no ped. button or ped. signal) then use Table 3-6 to determine the minimum pedestrian phase time; otherwise, use P_p = 0.0 s.

4 - Obtain from the Critical Volume Worksheet the critical volume that is associated with each movement. Only one phase combination (or row) should be used.

5 - Record the critical phase volume in all cells that correspond to the movements served.