Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-09/0-5629-1		

4. Title and Subtitle	5. Report Date
DEVELOPMENT OF A TRAFFIC SIGNAL OPERATIONS HANDBOOK	September 2008 Published: August 2009
	6. Performing Organization Code
7. Author(s) James Bonneson, Michael Pratt, and Karl Zimmerman	8. Performing Organization Report No. Report 0-5629-1
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135	10. Work Unit No. (TRAIS) 11. Contract or Grant No. Project 0-5629
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, Texas 78763-5080	13. Type of Report and Period Covered Technical Report: September 2006-August 2008 14. Sponsoring Agency Code

15. Supplementary Notes

Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.
Project Title: Best TxDOT Practices for Signal Timing and Detection Design at Intersections
URL: http://tti.tamu.edu/documents/0-5629-1.pdf

16. Abstract

The Texas Department of Transportation (TxDOT) operates thousands of traffic signals, both in rural areas and small cities. TxDOT's operation of these signals has served the state well over the years. However, regional differences in signal timing and detection design practice have evolved. These differences create operational inconsistencies and, possibly, sub-optimal performance. Good signal timing practices developed in some areas are not well documented or otherwise communicated to other areas. A comprehensive signal timing resource guide is needed to promote uniform, effective signal operation on a statewide basis.

This document summarizes the research conducted and the conclusions reached during the development of a *Traffic Signal Operations Handbook*. The handbook provides guidelines for timing traffic control signals at intersections that operate in isolation or as part of a coordinated signal system. The research conducted included a review of the literature, a survey of TxDOT engineers, an evaluation of alterative signal controller settings and detection designs. A spreadsheet was developed to accompany the *Handbook*. This spreadsheet automates several tasks involved in the development of a signal timing plan and is intended to facilitate implementation of the *Handbook* guidance.

17. Key Words	18. Distribution Statement
Signalized Intersections, Intersection Design, Intersection Performance, Traffic Signal Timing	No restrictions. This document is available to the public through NTIS: National Technical Information Service Springfield, Virginia 22161 http://www.ntis.gov

19. Security Classif.(of this report)	20. Security Classif.(of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	92	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

DEVELOPMENT OF A TRAFFIC SIGNAL OPERATIONS HANDBOOK

by

James Bonneson, P.E.
Senior Research Engineer
Texas Transportation Institute

Michael Pratt, P.E.
Assistant Research Engineer
Texas Transportation Institute

and

Karl Zimmerman, P.E.
Assistant Professor of Civil Engineering
Valparaiso University, Indiana

Report 0-5629-1
Project 0-5629
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Texas Department of Transportation
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Federal Highway Administration

September 2008
Published: August 2009

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data published herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) and/or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation. It is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was James Bonneson, P.E. #67178.

NOTICE

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

ACKNOWLEDGMENTS

This research project was sponsored by the Texas Department of Transportation and the Federal Highway Administration. The research was conducted by Dr. James Bonneson, Mr. Michael Pratt, and Dr. Karl Zimmerman. Dr. Bonneson and Mr. Pratt are with the Texas Transportation Institute. Dr. Zimmerman was with the Texas Transportation Institute at the time of his participation in the project, he is now with Valparaiso University in Indiana.

The researchers acknowledge the support and guidance provided by the Project Monitoring Committee:

- Mr. Larry Colclasure, Program Coordinator (TxDOT);
- Mr. Henry Wickes, Project Director (TxDOT);
- Mr. Don Baker (TxDOT);
- Mr. Adam Chodkiewicz (TxDOT);
- Mr. David Danz (TxDOT);
- Mr. Gordon Harkey (TxDOT);
- Mr. Derryl Skinnell (TxDOT);
- Mr. Dexter Turner (TxDOT); and
- Mr. Wade Odell, Research Engineer (TxDOT, Research and Technology Implementation Office).

The researchers also acknowledge the insight and advice provided by Mr. Kirk Barnes and Mr. Nader Ayoub during the development of several report chapters and appendices.

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CHAPTER 1. INTRODUCTION

OVERVIEW

The Texas Department of Transportation maintains and operates about 6000 traffic signals in Texas. TxDOT is supported in this activity by cities with a population of 50,000 or more. Specifically, TxDOT arranges by agreement to have these cities maintain the signals on the state highway system. Also, TxDOT sometimes arranges by agreement to have these cities maintain the signals on the access roads of controlled-access facilities.

Differences have appeared in the design and operation of traffic signals across the state. Some of these differences are due to the variation in geography, climate, population, traffic composition, and driver population that exists within the state. However, other differences are due to the limited amount of statewide guidance available on traffic signal operation.

As a result of limited statewide guidance, each district has tended to develop its own guidelines for signal operation. The result is that variations in signal operating philosophy have emerged among TxDOT districts. This flexibility has served Texas reasonably well over the years. However, the consequence of having different operating philosophies is a lack of uniformity among districts and possible sub-optimal signal operation. As traffic volume grows on Texas highways, sub-optimal performance of traffic signals may result in increased delay and fuel consumption.

A single statewide policy for signal timing settings and detection designs may be too limiting due to the differences in climate, terrain, and driver behavior across the state. However, a document that describes a range of effective settings and designs would allow traffic engineers to identify the "best" solution for his or her district's conditions. This approach is consistent with TxDOT's goal of ensuring uniform signal operations on a statewide basis, while giving local engineers the ability to make small adjustments (with an allowable range) to adapt the settings, design, or both to local conditions.

OBJECTIVE

The objective of this research was to develop traffic signal operations guidance suitable for statewide application and document it in a *Traffic Signal Operations Handbook*. This *Handbook* provides guidance on signal timing and detection design for isolated signalized intersections and interchanges as well as coordinated signal systems. The *Handbook* describes alternatives, discusses differences between these alternatives, and defines conditions where each alternative is most appropriate.

The primary intent of the *Handbook* is to summarize current signal operating practices and identify either the "best practice" or provide guidance where equally workable alternative practices exist. Where appropriate, research was conducted to resolve significant differences in practice and to fill knowledge gaps. This report documents the activities undertaken to: (1) synthesize best signal timing and design practices based on an extensive review of the literature and practitioner interviews, and (2) resolve differences in practice and fill knowledge gaps through original research.

Several elements of traffic signal design were outside the scope of this research. These topics include:

- signal justification (i.e., warrants and engineering studies),
- traffic control device design (e.g., signal head displays and mounting),
- signal infrastructure (e.g., poles, mast arms, foundations, etc.),
- plan preparation, and
- signal hardware maintenance.

RESEARCH APPROACH

A two-year program of research was developed to satisfy the project's stated objective. The research approach consists of eight tasks that represent a logical sequence of review, research, evaluation, and workshop development. These tasks are identified in the following list.

1. Evaluate state-of-the-practice.
2. Conduct TxDOT district interviews.
3. Develop *Handbook* outline and research plan.
4. Conduct research for *Handbook*.
5. Develop draft *Handbook*.
6. Solicit comments and revise draft *Handbook*.
7. Develop introductory workshop materials.
8. Prepare research report.

The research conducted in Tasks 1 through 4 and 8 is documented in this report. The findings from Tasks 5, 6, and 7 are reflected in the *Handbook* that was produced for this project.

CHAPTER 2. REVIEW OF STATE AGENCY SIGNAL TIMING PRACTICES

OVERVIEW

This chapter documents the findings from a review of the signal timing practices of several state departments of transportation (DOTs) in the United States. The documentation provided by these DOTs was reviewed to identify similarities and differences in signal timing practice among the state DOTs.

State DOT signal-related documents (e.g., traffic signal manuals, standard drawings, policies, etc.) were reviewed to identify the range of guidance that is being provided for signal timing. The review was also extended to detection design. Only readily available documents were referenced (e.g., documents available on the Internet). No attempt was made to be comprehensive in the review of documents, and only fifteen states were represented in the review. These states are listed in Table 2-1.

Table 2-1. State DOT Document Sources and Topics Reviewed.

Signal-Related Document Source	Topics Reviewed							
	Change Period		Left-Turn Phasing	Minimum Green	Maximum Green	Flashing Operation	Pre-emption	Detection
	Yellow Change	Red Clear						
Arizona (*1*)	✔	✔	✔	--	--	✔	✔	✔
California (*2*)	✔	✔	✔	--	--	--	✔	✔
Connecticut (*3*)	✔	✔	✔	--	--	✔	✔	✔
Florida (*4*)	✔	✔	✔	--	--	✔	--	✔
Idaho (*5*)	✔	✔	✔	✔	✔	✔	✔	✔
Indiana (*6*)	✔	✔	✔	✔	--	--	--	✔
Maryland (*7*)	--	--	--	--	--	--	✔	✔
Minnesota (*8,9*)	✔	✔	✔	✔	✔	✔	✔	✔
Ohio (*10*)	✔	✔	--	--	--	✔	✔	✔
Oregon (*11,12,13*)	--	--	✔	--	--	✔	✔	✔
South Dakota (*14*)	✔	✔	✔	--	--	✔	✔	✔
Texas (*15*)	--	--	--	--	--	✔	✔	--
Utah (*16,17*)	✔	✔	--	✔	✔	--	✔	✔
Virginia (*18*)	--	--	✔	--	--	--	--	✔
Washington (*19*)	--	--	✔	--	--	--	✔	✔

Note:
"--" information on topic not available.

The review of state DOT documentation revealed the availability of guidance on many different areas of signal timing and detection design. However, the search was narrowed to allow focus on key areas for which a wider range in practice was identified. These key focus areas are:

- change period (i.e., yellow change and red clearance intervals),
- left-turn phasing,
- minimum green,
- maximum green,
- flashing operation,
- preemption, and
- detection.

The reference numbers in column 1 of Table 2-1 identify the specific documents that were reviewed. The checkmark (✔) in subsequent columns describes the topics covered in the corresponding document. Only two documents addressed every topic and no single topic was addressed by every state.

The fact that a document is listed in Table 2-1 as not providing guidance on a particular topic does not mean that particular topic is not addressed by that state. Instead, it most likely means that the particular guidance on this topic is determined by local, regional, or district engineers (instead of being specified on a statewide basis). This flexibility reflects differences in climate, topography, and driver behavior across the individual state. In some cases, the lack of statewide guidance on a topic may mean only that it is not in a publicly accessible form (e.g., internal memoranda).

Table 2-1 provides some indication of the considerable variation in the content of state documentation for traffic signal timing and detection design. Many of these documents include information that describes how signal projects are handled, construction details or specifications, signal warranting information, or traffic engineering study procedures. This information is outside the scope of the review for this research.

The remainder of this chapter summarizes the findings from the review of the state DOT documents identified in Table 2-1. The summary is divided into three parts. The first part addresses signal controller timing and signal phasing. The second part addresses signal settings that are typically used for special applications. The last part addresses detection design and operation.

SIGNAL CONTROLLER TIMING AND PHASING

This part of the chapter summarizes state DOT practice related to the following five topics:

- minimum green,
- maximum green,
- yellow change and red clearance intervals,
- left-turn operational mode, and
- left-turn phasing.

Minimum Green

For actuated phases, the minimum and maximum green settings are important for efficient signal operation (*20*). A long minimum green can be inefficient when queues are short. In contrast, a short minimum green may cause occasional premature phase terminations due to variation in queue discharge time.

Five DOT documents identified recommended values for the minimum green setting. These values are listed in Table 2-2 and should be regarded as the shortest allowed minimum green setting. Idaho DOT recommends a minimum green setting of 4 s to provide "snappy" operation (i.e., rapid response to traffic demands). On the other hand, Minnesota, South Dakota, Utah, and Indiana DOTs recommend major-road minimum green settings on the order of 8 to 20 s. South Dakota, Utah, and Indiana DOTs also recommend a minimum green setting equal to the sum of the pedestrian reaction time and pedestrian clearance time, if pedestrians are present.

Table 2-2. Minimum Green Setting.

State DOT	Applicable Phases	Minimum Green Setting, s [1]
Idaho	All	4
Indiana	Through movement	10 to 20
	Left-turn movement	5 to 7
Minnesota	Major-road through movement, 40 mph or less	15
	Major-road through movement, 45 mph or more	20
	Minor-road through movement	10
	Protected-only left-turn movement	7
	Protected-permissive left-turn movement	5
South Dakota	Major-road through movement	12
	Minor-road through movement	7
	Left-turn movement	4
Utah	High-speed through movement (more than 40 mph)	20
	Crossing and minor arterials through movement	15
	Minor road movement	4 to 5
	Left-turn movement	4 to 5

Note:
1- Values shown are the shortest minimum green setting allowed; the actual minimum green used may be longer.

Minnesota, Utah, Indiana, and South Dakota DOTs also identify recommended minimum green settings for minor-road and left-turn movement phases. These values are shown in Table 2-2. All four DOTs indicated that these minimum green values were based on consideration of driver expectancy. Minnesota DOT indicates that minimum green can vary depending on whether a protected-only or protected-permissive left-turn mode is used. This distinction may reflect the additional turning opportunities that are allowed by the protected-permissive mode.

Maximum Green

A long maximum green setting allows time for the controller to ensure effective queue clearance and, if advance detection is used, to ensure a safe phase termination. However, an unnecessarily long maximum green setting can result in excessive delay to other movements.

Idaho, Minnesota, and Utah DOTs use the following equation for calculating the maximum green setting for each phase:

$$G_{max} = Nominal\ \ Green\ \ Time \times Adjustment\ \ Factor \tag{1}$$

where,
G_{max} = maximum green setting, s.

The adjustment factor is used to account for random variation in vehicle arrivals each cycle. The "nominal green time" and "adjustment factor" variables used in this equation vary among the DOTs. The rationale for determining the nominal green time and the recommended factor values are listed in Table 2-3.

Table 2-3. Maximum Green Variables.

State DOT	Nominal Green Time, s	Adjustment Factor
Idaho	Average queue clearance time	1.2 to 1.3
Minnesota	3.0 + 2.1×(average number of vehicles in queue per cycle)	1.5
Utah	Minimum-delay green interval duration	1.25 to 1.50

Minnesota DOT and Utah DOT also provide typical ranges of maximum green settings. These values are listed Table 2-4. It is interesting to note that Minnesota DOT's document states that the maximum green may be as large as 120 s for a major-road through movement.

Table 2-4. Maximum Green Setting.

State DOT	Volume-Density Feature Used?	Applicable Phases	Maximum Green Setting, s
Minnesota	No	Left-turn movement	10 to 45
		Minor-road through movement	20 to 75
		Major-road through movement	30 to 120
	Yes	Minor-road through movement	30 to 75
		Major-road through movement and speed less than 45 mph	45 to 120
		Major-road through movement and speed of 45 mph or more	60 to 120
Utah	not specified	not specified	15 to 60

Change Period

The change period provides the time necessary to safely end one vehicle phase and begin another. The change period consists of the yellow change interval and the red clearance interval. These two intervals are individually discussed in this section.

Yellow Change Interval

The yellow change interval provides a warning to drivers that the phase is ending and right-of-way is being transferred to another phase (*21*). Most states use the following equation to calculate the yellow change interval (*22*):

$$y = t + \frac{1.47\ v}{2(a + 32.2g)} \tag{2}$$

where,

- y = yellow change interval duration, s;
- t = perception-reaction time (use 1.0 s), s;
- v = approach speed, mph;
- a = deceleration rate (use 10 ft/s^2), ft/s^2; and
- g = approach grade, uphill grade is positive (= percent grade/100), ft/ft.

The *Manual on Uniform Traffic Control Devices* (*MUTCD*) states that the yellow change interval should be between 3 and 6 s (*21*).

The Florida and Idaho DOTs recommend yellow change interval durations that are different from that computed by Equation 2. These durations are shown in Table 2-5. It is noted that Idaho DOT presents both the Equation 2 and the information in Table 2-5 as methods to determine yellow intervals. The Idaho DOT's manual states that the values in Table 2-5 are intended to "present...the same yellow [change] interval at comparable intersections" (*5*).

Two DOT documents provide additional guidance about yellow change intervals. Connecticut DOT's signal manual states that yellow change intervals should not normally be longer than 5 s (*3*). This type of restriction is recognized by other agencies and reflects a concern about driver disrespect for long yellow change intervals (*20*). Minnesota DOT uses Equation 2, but rounds the results up to the nearest 0.5 s, as long as the result does not exceed 6 s.

Table 2-5. Yellow Change and Red Clearance Intervals.

Approach Speed, mph	Equation 2		Florida DOT		Idaho DOT	
	Yellow Change, s	Red Clearance, s [1]	Yellow Change, s	Red Clearance, s	Yellow Change, s	Red Clearance, s [1,2]
25	3.0	2.3	3.5	1.0	3.2	2.1
30	3.2	2.0	3.5	1.0	3.2	2.0
35	3.6	1.7	4.0	1.0	3.2	2.1
40	3.9	1.5	4.0	1.0	4.0	1.4
45	4.3	1.3	4.3	1.0	4.0	1.6
50	4.7	1.2	4.7	1.0	4.0	1.9
55	5.0	1.1	5.0	2.0	4.0	2.1
60	5.4	1.0	5.4	2.0	5.0	1.4
65	5.8	0.9	5.8	2.0	5.0	1.7

Notes:
1- Assumes intersecting road has five lanes (60 ft pavement), 6-ft stop line setback, and a 20-ft vehicle length.
2- Red clearance interval is optional at speeds below 40 mph and required at higher speeds. It should not exceed 2.0 s.

Red Clearance Interval

As with the yellow change interval, most states use the following equation to calculate the red clearance interval (*22*):

$$r = \frac{W + L_v}{1.47\ v} \quad \textbf{(3)}$$

where,
r = red clearance interval duration, s;
W = width of intersection, ft; and
L_v = length of vehicle (use 20 ft), ft.

The red clearance interval is recognized as an optional interval in the *MUTCD* (*21*). For this reason, its use varies widely among the DOTs. California DOT's manual does not require the use of a red clearance interval and gives no equation for calculating its duration. This manual indicates that a red clearance interval should be applied if engineering judgment indicates it is needed, and if used, the red clearance interval should be between 0.1 s and 2 s.

Florida DOT's recommended red clearance intervals are shown in Table 2-5. These intervals *increase* with speed, unlike the values obtained from Equation 3.

Idaho DOT does not require a red clearance interval for approach speeds below 40 mph. Where used, the red clearance interval should be between 0.5 s and 2.0 s, adjusted using engineering judgment. This DOT's guidelines indicate that the red clearance interval is computed by adding the values from Equations 2 and 3, and then subtracting the yellow change interval. The yellow change intervals recommended by Idaho DOT are shown in column 6 of Table 2-5. The red clearance values computed in this manner are listed in the last column of Table 2-5.

Utah DOT uses a red clearance interval of 1.5 s, regardless of approach speed or crossroad width. In contrast, Connecticut DOT uses Equation 4 to calculate the red clearance interval.

$$r = \frac{D_c}{1.47\ v_c} - \frac{D_e}{1.47\ v_e} + t_c \qquad \textbf{(4)}$$

where,

t_c = time for clearing vehicle to clear conflict point (use 1.0 s), s;
D_c = distance from clearing vehicle's stop line to the farthest conflict point, ft;
v_c = clearing vehicle speed, mph;
D_e = distance from entering vehicle's stop line to the conflict point associated with D_c, ft; and
v_e = entering vehicle's speed, (use 15 mph), mph.

Equation 4 estimates the minimum separation time between two conflicting vehicles as measured at the conflict point where their paths cross in the intersection conflict area. A conflict point is any point where the last vehicle clearing the intersection could collide with the first vehicle or pedestrian entering the intersection. The "critical" conflict point is defined by the conflicting movement pairs that require the longest separation time for the subject phase.

Left-Turn Operational Mode

There are three operational modes for the left-turn movements at an intersection. They include: permissive mode, protected mode, and protected-permissive mode. The DOT documents reviewed indicated a range of guidelines being used for determining the most appropriate mode at a particular intersection. These guidelines are summarized in this section.

Protected or Protected-Permissive Mode

This subsection describes guidelines used by several state DOTs to determine when a left-turn phase is appropriate. Many of these guidelines do not make a distinction between the protected and protected-permissive modes. The Florida DOT manual is an exception. It indicates that the protected-permissive mode is preferred. Common considerations in the formulation of left-turn phase selection criteria are provided in the following list:

- product of left-turn and oncoming through volumes (Arizona, California, Oregon, Virginia),
- left-turn volume (South Dakota, Arizona, California, Indiana),
- delay to left-turn vehicles (Arizona, California, Indiana), and
- crash history (Arizona, California, Idaho, Minnesota, Oregon, Indiana, Virginia).

Some state DOTs have identified additional criteria that are not provided in the previous list. For example, California DOT recommends trying other options first to avoid the use of a left-turn phase. Florida and Connecticut DOTs require left-turn phases on both road approaches, even if only one direction appears to be justified. In contrast, Idaho DOT's manual states that a left-turn phase should only be used on the intersection approach where it is justified.

The individual states do not agree on the actual thresholds to be used for the selection criteria. For example, Virginia DOT uses a product threshold of 50,000; California DOT uses a value of

100,000; Oregon DOT uses a range from 50,000 to 300,000 depending on the lane configuration and phasing type; and Arizona DOT uses a range from 50,000 to 225,000, depending on the lane configuration and whether the intersection is urban or rural.

Protected Left-Turn Phasing

Protected left-turn phasing is the most restrictive operational mode because it does not allow permissive left turns. For this reason, its inclusion in the signalization tends to increase the delay to other intersection movements. On the other hand, it provides a safe left-turn operation because all conflicting movements face a red indication while the subject left-turn phase is served.

Some common guidance emerges when comparing the state DOT guidelines for using protected left-turn phasing. This guidance includes the following considerations:

- turning across three or more oncoming through lanes (Arizona, Florida, Oregon, Virginia),
- approach speed exceeds 45 mph (Arizona, Florida, Minnesota, Oregon, Virginia),
- left-turn sight distance restriction (Arizona, Florida, Minnesota, Oregon, Virginia),
- dual left-turn lanes (Arizona, Connecticut, Florida, Oregon, Virginia),
- crash problem with protected-permissive (Arizona, Florida),
- crash problem was reason to add left-turn phasing (Idaho, Oregon, Virginia),
- lead-lag left-turn phasing (Florida, Minnesota, Oregon if flashing arrow not available),
- unusual geometry (Florida, Minnesota, Oregon, Virginia), and
- left-turn volume (Oregon, Indiana, Virginia).

In addition to the considerations in the preceding list that are attributed to Florida DOT, this agency also recommends protected left-turn phasing when: (1) the left-turn lanes are 10 ft or more from the through lanes and separated by islands, or (2) the subject left-turn phase is the leading left-turn phase of lead-lag phase sequence.

Left-Turn Phasing

A left-turn phase is described as "leading" or "lagging" based on whether it occurs before or after the phase serving the opposing through movement in the signal cycle. If both left-turn phases lead, then the sequence is called "lead-lead." Alternatively, if both left-turn phases lag, then the sequence is called "lag-lag." When one left-turn phase leads and the other phase lags, the sequence is called "lead-lag." If the left-turn movement is provided a green-arrow indication concurrently with the adjacent through movement, then the phase sequence is described as split phasing.

The state DOT documents do not describe guidelines that indicate when to use lead-lead, lead-lag, or lag-lag phasing. However, several documents describe general guidelines indicating when split phasing should be considered. Moreover, they acknowledge a potential safety issue when lead-lag or lag-lag phasing is used with the protected-permissive mode. This issue relates to a "yellow trap" that occurs. Guidelines for split phasing and a discussion of the yellow trap are provided in the remainder of this section.

Split Phasing

With split phasing, the green indications for all of the movements on a single intersection approach begin and end simultaneously. Some state DOTs, such as Florida and Arizona, describe guidelines for justifying split phasing. Arizona DOT's guidelines indicate the use of split phasing when:

- the intersection has either offset legs or overlapping turning paths,
- heavy left-turn volume and adjacent through volume,
- no separate left-turn lane on approach, or
- shared left-turn and through lanes.

Florida DOT's requirements are similar to those used by Arizona DOT.

Yellow Trap

If the protected-permissive mode is used with lead-lag or lag-lag phasing, then the yellow trap (or left-turn trap) problem may occur for one or both of the left-turn movements. This problem stems from the potential conflict between left-turn vehicles and oncoming vehicles at the end of the adjacent through phase. Of the two through movement phases serving the subject road, the trap is associated with the first through movement phase to terminate and occurs during this phase's yellow change interval. The left-turn driver seeking a gap in oncoming traffic during the through phase first sees the yellow ball indication, then incorrectly assumes that the oncoming traffic also sees a yellow indication; the driver then turns across the oncoming traffic stream without regard to the availability of a safe gap.

The yellow trap is illustrated in Figure 2-1. During Stage 1, the southbound and northbound left-turn movements are operated in the permissive mode during their respective through phases. During Stage 2, the southbound left-turn and through signal heads display a yellow change interval. However, the northbound left-turn and through heads continue to display a green indication. This condition can create a yellow trap for the southbound left-turn driver if he or she assumes that northbound drivers are also viewing a yellow indication and preparing to stop. Following this incorrect assumption, the southbound left-turn driver attempts to complete the left turn maneuver even though he or she is exposed to oncoming traffic.

The state DOT manuals differ as to whether or not they specifically mention or describe the yellow trap. Nine of the 15 state DOT manuals mention the yellow trap. Six of these nine manuals suggest countermeasures. These countermeasures are summarized in the following paragraphs.

The yellow-trap countermeasure identified in the Washington State DOT manual is the use of a lead-lead phase sequence with phase recall for a crossroad through phase. This technique insures that a crossroad phase always times after the subject road through phase and before its left-turn phase. However, the manual also acknowledges that this approach may unnecessarily delay the major-road movements during periods of low crossroad volume.

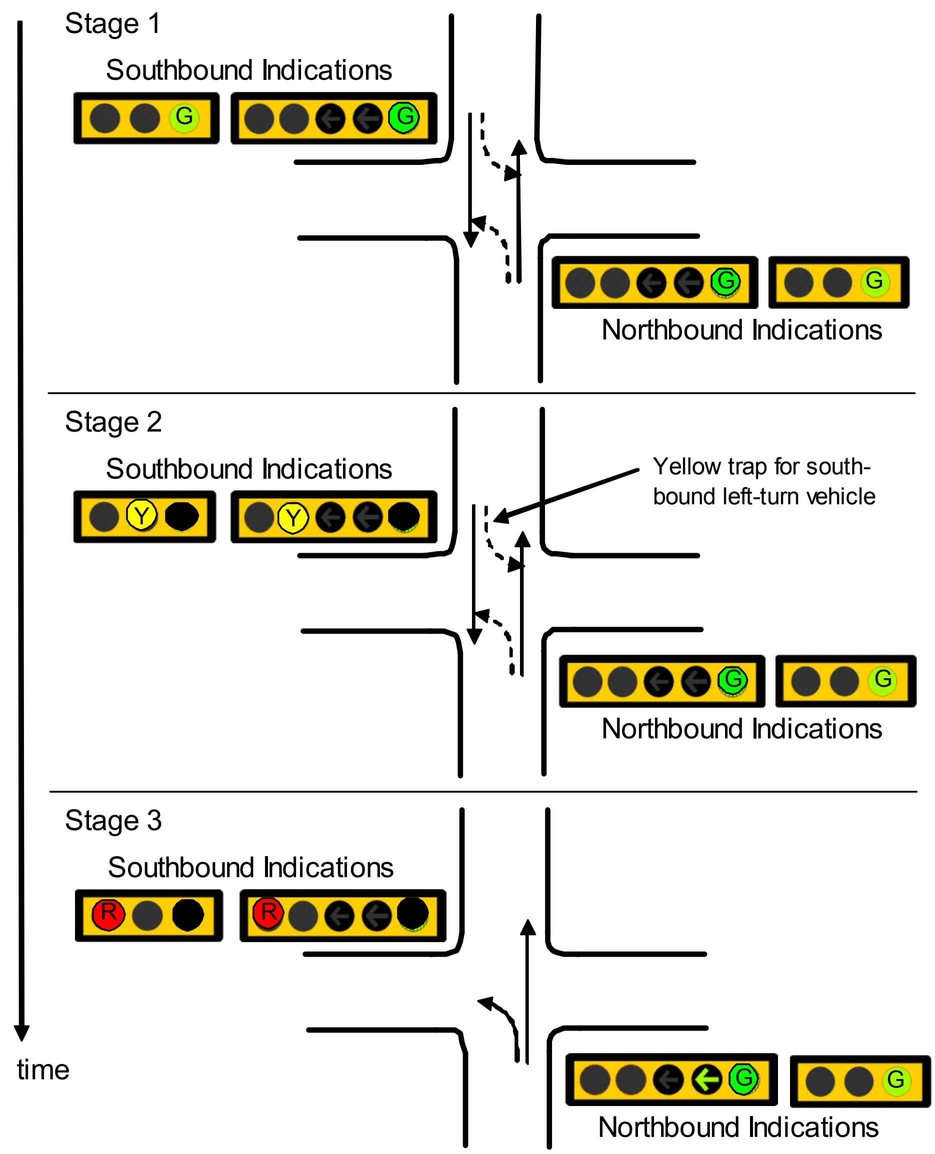

Figure 2-1. Demonstration of Yellow Trap with Lead-Lag Phasing.

The yellow trap countermeasure identified in the Oregon DOT manual is based on the use of a permissive left-turn indication that is assigned to a controller overlap associated with the *opposing* through movement phase. The protected left-turn indications are assigned to the left-turn phase output. This operation eliminates the yellow trap by maintaining the permissive left-turn indication (as opposed to having it show a yellow indication) when the adjacent through phase ends. A flashing yellow arrow is used for the permissive left-turn indication by Oregon DOT.

The aforementioned countermeasure is also used by the City of Dallas, Texas where it is referred to as "Dallas Phasing" (*23*). The only differences are that (1) the overlap is associated with both the adjacent and opposing through movement phases, and (2) a louvered green ball indication is used for the permissive left-turn indication, as shown in Figure 2-2. The louvers are necessary to prevent the indication from being seen by drivers in the adjacent through lanes (*21,* Section 4D.08).

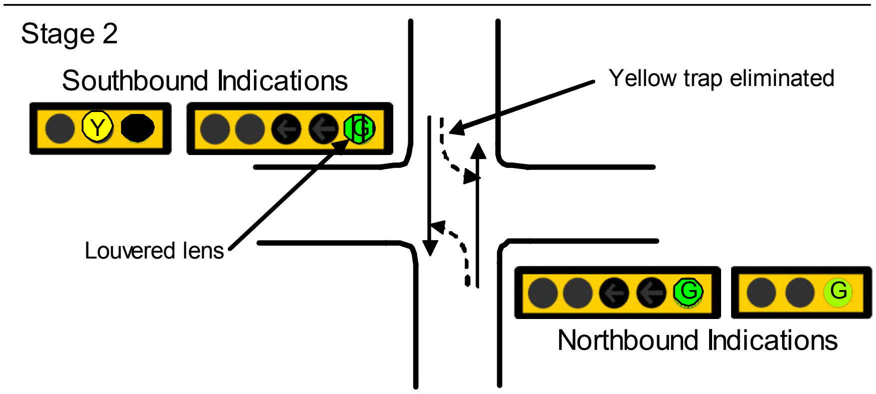

Figure 2-2. Dallas Phasing to Eliminate Yellow Trap.

Other state DOTs (e.g., Florida DOT and Connecticut DOT) require that the leading left-turn phase of a lead-lag phase sequence operate in the protected mode. While this countermeasure eliminates the yellow trap, it eliminates some of the efficiency gains afforded by the protected-permissive mode.

ADVANCED SIGNAL TIMING FUNCTIONS

Advanced signal timing functions address situations that are outside the normal stop-and-go operation of a traffic signal. These functions include variable initial, gap reduction, flashing operation and preemption. Other possible functions include time-of-day operation and coordination. However, due to considerable differences in signal timing practices between the states, only flashing operation and preemption are addressed in this document.

Variable Initial Settings

In situations where detection is not provided at the stop line, the variable initial feature can be used to ensure that the green interval is sufficiently long to discharge the stopped queue. A detector located upstream of the intersection is used to count the number of vehicle actuations during the yellow and red indications, and then compute the green time needed (referred to as "added initial") to clear the waiting queue.

At least three of the state DOT documents reviewed (i.e., Connecticut, Virginia, Minnesota) recommended the use of the variable initial settings. However, only Minnesota DOT's manual provides specific guidance for determining the variable initial settings.

Gap Reduction Settings

The Minnesota, Utah, and Indiana DOTs use the gap reduction feature to minimize phase termination by extension to the maximum limit (i.e., max-out). This feature is also used when advance detection is provided but stop line detection is not provided. Utah DOT's manual recommends the use of this feature whenever the associated phase passage time is greater than 3.0 s.

Controller settings associated with the gap reduction feature are: time before reduction, time to reduce, and minimum gap. Gap reduction settings recommended by Utah DOT and Minnesota DOT are shown in Table 2-6. Utah further recommends that the last car passage setting is off when the gap reduction feature is used with advance detection on high-speed approaches.

Table 2-6. Gap Reduction Settings.

State DOT	Setting	Value
Minnesota	Passage time	3.0 to 5.0 s
	Time before reduction	one-third of maximum green
	Time to reduce	one-third of maximum green
	Minimum gap	at least 2.0 s
Utah	Passage time	2.5 to 4.0 s, with 3.0 s typical
	Time before reduction	minimum green
	Time to reduce	(maximum green - minimum green)/2
	Minimum gap	2.0 to 3.0 s

Flashing Operation

In flashing operation, a traffic signal's operation changes from its normal green-yellow-red cyclic pattern to a flashing single-indication pattern. Flashing operation is used to indicate a malfunction or to reduce the level of intersection control during periods of low volume. There are two types of flashing operation: planned and unplanned. These two types are discussed below.

Planned Flashing Operation

Planned flashing operation is called by the following names in the state DOT documents: "programmed," "nighttime," and "off-peak." In each case, flashing operation is an intentional, planned event that is set to occur regularly at certain times of day. The motivation for this type of operation is to reduce delay and stops when traffic volume is low.

When planned flashing operation is used, 12 of the 15 state DOT documents reviewed use a "yellow-red" flash pattern, where the major-road through movement indications flash yellow and all of the minor movement indications flash red. Three state DOTs differed from this practice, they include Idaho DOT, South Dakota DOT, and Oregon DOT. The Idaho and South Dakota DOT manuals recommend that all signal indications on an approach should flash the same color (i.e., the major-road left-turn indications should flash yellow if the adjacent through movement indications flash yellow). In contrast, the Oregon DOT manual uses "all-red" flash (i.e., all indications flash red) for all approaches as its standard flash mode. However, if the major-road to minor-road volume ratio is greater than 4:1, then yellow-red flash is allowed and the major-road left-turn phase indications may flash red.

Each state DOT manual that discusses planned flashing operation uses somewhat different criteria for this operation. For example, the Connecticut DOT manual states that planned flash may be used during hours that the signal does not meet the signal warrants. The Texas DOT manual states that fully actuated intersections should not be placed in flash unless all other signals in the area operate in flash. At the other extreme, the Ohio DOT manual cites the following criteria should be considered before using planned flashing operation:

- No sight distance deficiencies exist on any leg of any approach.
- The major-road volume is less than 200 veh/h, although flash may be allowed at higher volumes if the major-road to minor-road volume ratio is greater than 3:1.
- Do not begin flashing operation until one hour after local drinking establishments close.
- Keep "some" signals in stop-and-go operation to prevent a "speedway effect."
- Monitor the crash history after implementation and revert to stop-and-go operation if:
  - three or more right-angle crashes occur in one year, or
  - two right-angle crashes per million entering vehicles during flashing operation, if 3 to 5 right-angle crashes observed, or
  - 1.6 right-angle crashes per million entering vehicles during flashing operation, if 6 or more right-angle crashes observed.

Florida DOT's guidelines are similar to Ohio DOT's, although less extensive. The Idaho DOT's manual allows planned flashing operation if:

- adequate gaps exist for cross traffic,
- intersection crash history does not show a tendency for right-angle collisions,
- there is a high ratio of major to minor volumes,
- crashes were not the reason the signal was installed,
- sight distance is not a problem on any intersection approach, and
- distraction and glare from flashing operation itself will not be a problem.

Generally, all state DOT documents agree that all intersections in an area that are to be placed in planned flash are to be put in flash simultaneously and that the pedestrian indications should be dark during flash.

Unplanned Flashing Operation

Unplanned flash occurs during a malfunction of the signal operating equipment. The purpose of unplanned flash is to provide safe intersection operation during periods where the signal is inoperable or functioning improperly. In most state DOT documents, unplanned flashing operation must use "all-red" flash. The Connecticut and Minnesota DOT documents allow "yellow-red" flash during unplanned flashing operation.

Other Uses of Flashing Operation

When starting a new traffic signal, a common practice is to use flash for a period of time prior to beginning normal signal operations. The idea is to get drivers used to having the signal at that particular intersection. However, the Idaho DOT signal manual states that using flashing operation during signal start-up is "no longer preferred," and recommends only using temporary warning signs for two weeks after signal start-up (*5*).

The Minnesota DOT manual states that if an intersection is in flash, any advance warning devices such as intersection warning signs should also flash during that time.

Preemption

Preemption is used with traffic signals to stop the current timing plan and implement an alternative plan that provides extended service for another type of movement, including a non-roadway movement. Usually, safety is the prime motivation for using signal preemption. This section discusses the types of preemption sources that are accommodated and the operation of the traffic signal during preemption.

Railroad Preemption

The various state DOT documents did not agree about whether the pedestrian change interval could be shortened by rail preemption. Most state DOTs do not allow the pedestrian change interval to be shortened by rail preemption. The Oregon DOT manual specifically states that a pedestrian change interval must occur prior to track clearance.

A survey of engineers in each of TxDOT's 25 districts indicated that only about 4 percent of traffic signals statewide have railroad preemption. Of the signals with railroad preemption, about 42 percent have advance preemption. Pedestrians are not present at intersections with preemption in 20 percent of the districts. Engineers with the remaining 80 percent of districts were asked whether they truncate the pedestrian Walk or Flashing Don't Walk intervals. Twenty percent of the districts truncate just the Walk interval and 46 percent of the districts truncate both intervals. The districts that truncate these intervals do so because the intersection does not have advance preemption. These districts also use flashing operation during a railroad preempt. As soon as flashing operation begins, the pedestrian heads go dark and stay that way until the preempt ends.

Emergency Vehicle Preemption

Several state DOT documents mentioned emergency vehicle preemption, but differed on provisions for it. The Oregon DOT manual requires the addition of emergency preemption equipment with all new traffic signals. In contrast, the Ohio DOT is not convinced that emergency preemption is worth the time and expense because no studies have shown that it necessarily improves response times or reduces emergency vehicle conflicts. Ohio DOT only provides emergency vehicle preemption by special request. Where used, emergency preemption is second only to railroad preemption in priority.

Transit and Light Rail Preemption

A few state DOT documents mentioned transit preemption. The Washington State DOT does not install transit preemption but will allow it if installed by transit agencies. In Washington State, light rail preemption is handled similarly to emergency vehicles rather than heavy rail. In Oregon, light rail preemption may be exempt from heavy rail requirements. Oregon bus priority only modifies existing green times rather than changing phasing.

DETECTION DESIGN AND OPERATION

Most of the state DOT documents reviewed indicated a preference for inductive loop detectors. The inductive loop has proven to be reliable, although installation can be inconvenient and the loops are vulnerable to damage due to pavement deterioration and shift. Only the Maryland DOT manual specifically states a preference for video detection. Most of the DOT documents recognize the use of video detection when loop detection is not viable.

Each state DOT's standard detection layout is unique and reflects differences in operational goals, detection technology, climate, and pavement design. Most state DOTs indicate a preference for presence detection at the stop line. Some state DOT manuals identify standard detection designs for high-speed intersection approaches, while others do not acknowledge the need for advance detection. These differences in design philosophy result in detection designs that are difficult to generalize and difficult to compare. At this time, specification of standard detection designs appears to be a low priority among the state DOTs.

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CHAPTER 3. ALTERNATIVE DETECTION DESIGNS FOR HIGH-SPEED APPROACHES

OVERVIEW

This chapter describes an evaluation of alternative detection designs used by TxDOT for high-speed intersection approaches. Detection for high-speed approaches typically consists of multiple advance detectors that are configured to extend the green indication until the approach is clear of vehicles. Research has shown that there is a length of roadway on every intersection approach in which drivers are collectively indecisive about whether to stop or go at the onset of the yellow indication. Advance detection is used to clear this "indecision zone" and thereby, reduce the potential for rear-end crashes and red-light violations.

The objective of this research was to evaluate alternative high-speed detection designs used in Texas and to identify the most effective design configurations. In this regard, the detection design for a high-speed approach includes specification of the following factors:

- number of advance detectors,
- use of a stop line detector, including:
  - whether it uses a detector channel separate from that used for the advance detectors,
  - whether it is functionally disabled after queue clearance,
- distance between each detector and the stop line,
- length of each detector,
- use of presence or pulse mode detection,
- use of locking or non-locking memory,
- duration of controller passage time (i.e., extension), and
- use of supplemental call extension or delay for the detector channel input.

This chapter consists of three main parts. The first part of this chapter identifies the various detection designs used by the various TxDOT districts for high-speed approaches. The second part summarizes the evaluation procedure. The last part describes the findings from the evaluation of alternative designs.

EXISTING DETECTION DESIGNS

This part of the chapter summarizes the inductive-loop-based detection designs used in ten TxDOT districts for through movements on high-speed intersection approaches. It is believed that these designs are representative of those used by the other 15 districts. The first section describes the detection layout of these designs. The second section describes the controller settings typically used with these designs. The last section summarizes the observations from this review and identifies the issues that need to be resolved.

Detection Layout

The detection layout used by 10 TxDOT districts is identified in the top half of Table 3-1. The layout used by eight districts is listed in column 4. It has three advance detectors when the design speed is 55 mph or more and two advance detectors for design speeds of 45 and 50 mph. These detectors are more distant from the stop line for the higher design speeds. The stop line detector is 40 ft long and is wired to one of two detector channels assigned to the phase that serves the through movement. The second detector channel is assigned to the set of advance detectors. The controller settings listed in this table are discussed in the next section.

Table 3-1. Detection Layout Used in Several TxDOT Districts.

Category	Design Speed	Design Element	TxDOT District		
			8 Districts	District P	District K
Detection Layout	70	Distance from the stop line to the upstream edge of the advance detector, ft (Note: The number of distances listed indicates the number of advance detectors. All advance detectors are 6 ft in length)	600, 475, 350	?	606, 481, 356
	65		540, 430, 320	?	546, 436, 326
	60		475, 375, 275	320, 220, 140, 80	481, 381, 281
	55		415, 320, 225		421, 326, 231
	50		350, 220		356, 226
	45		330, 210	?	296, 181
	45 to 70	Stop line detector length, ft	40	not used	40
	45 to 70	Advance detector lead-ins wired to separate channel from stop line detectors	Yes	not applicable	Yes
Controller Settings	70	Passage (extension) time, s	1.2	?	1.0 to 1.5 [1]
	65		1.2	?	
	60		1.4	1.5 to 2.0 [1]	
	55		1.2		
	50		2.0		
	45		2.0	?	
	45 to 70	Detection mode	Presence	Presence	Presence
	45 to 70	Detector memory	Nonlocking	Locking	Nonlocking
	45 to 70	Stop line detector channel extend setting, s	0.5 s	not applicable	not used
	45 to 70	Stop line detector operation (deactivated or continuously active) [2]	Deactivated after gap out	not applicable	?

Notes:
1 - Passage time is adjusted on an intersection-by-intersection basis within the stated range.
2 - Stop line detector operation is "deactivated" if it is disconnected after its detector channel extend timer times out. It is reconnected after the green interval terminates (see Special Detector, Operation Mode 4 in Eagle controller).
? - Data were not provided by the district.

The detection layout for District K is very similar to that used by the other eight districts. In contrast, the layout used by District P is quite different from the other districts and is consistent with a layout used by many TxDOT districts prior to about 1997 (*1*). It addresses design speeds in the range of 50 to 60 mph and does not require a stop line detector.

Controller Settings

The controller settings associated with the most commonly used detection layout are listed in column 4 of Table 3-1. A unique passage time is established for each design speed and detector layout. The detectors operate in the presence mode with nonlocking memory. The use of separate channels for the stop line and advance detectors allows the stop line detector to be electrically disconnected by the controller when the detector's extend timer times out. When this feature is enabled, it deactivates the stop line detector once the queue has been served so that the advance detectors can efficiently search for a safe time to end the phase.

The controller settings for Districts P and K are listed in columns 5 and 6, respectively, of Table 3-1. These designs do not include a stop line detector. For District P, the lack of a stop line detector is offset by the use of locking memory such that a vehicle arriving during the red indication is guaranteed to receive a green indication.

Observations

The information in Table 3-1 indicates that most districts are using the same detection layout. The layout used by District P does not appear to be widely used and is not fully specified. In fact, a before-after evaluation of the District P design and the "8 Districts" design was conducted by Middleton et al. (*1*). They found that the "8 Districts" design resulted in fewer vehicles being caught in the indecision zone and fewer vehicles running the red indication.

A comparison of the controller settings used by the "8 Districts" with those used by District K indicates some difference in strategy. Specifically, there appears to be a preference for flexibility in the selection of passage time in District K. Also, District K, and many other districts interviewed, indicated that they do not use "deactivated" stop line detector operation.

EVALUATION FRAMEWORK

This part of the chapter describes the framework used to evaluate the operational and safety performance of alternative controller settings for through movement detection on high-speed intersection approaches. It identifies the recommended performance measures and the methods used to compute these measures.

Recommended Performance Measures

The recommended operational performance measures include control delay and max-out probability. Control delay is computed for all intersection movements and then used to compute the overall average intersection delay. Desirably, a detection design will minimize intersection delay. This delay can be computed using the methodology described in the *Highway Capacity Manual* (*2*).

Max-out probability represents the likelihood that the green interval will extend to its maximum limit. When a phase is terminated in this manner, it is likely that vehicles will be on the intersection approach at the onset of yellow. These vehicles will undergo a lengthy delay waiting for the next green indication. They are also more likely to be involved in a rear-end crash should they decide to stop and the following driver decide to continue through the intersection. With regard to this latter behavior, max-out can also be considered to be a safety performance measure. The max-out probability can be computed using the procedure described by Bonneson et al. (*3*).

The safety performance of each scenario can be evaluated in terms of its detection coverage. This measure is an indication of the extent to which the detection design minimizes the probability of one or more vehicles being in the indecision zone at the onset of the yellow indication. It is computed for the distribution of vehicle speeds and volumes during the design hour. The remainder of this section describes a procedure for computing detection coverage.

Detection Coverage

The detection design for a high-speed intersection approach should provide efficient operation and safe phase termination. Safe termination is achieved when an advance detection design is used to monitor the indecision zone during the green indication and extend the green when one or more vehicles are in this zone. Practical considerations dictate that such designs include only one, two, or three points of detection within the zone. Passage time is then used to "carry" a vehicle from one detector to the next until it is clear of the zone.

An effective detector design is one that provides an optimal balance between safety and operations. For example, if a short passage time is used, it becomes easier to end the major-road phase by gap out, resulting in a shorter average cycle length and reduced delay to minor-road vehicles. However, a short passage time is also more likely to result in a gap out when there are vehicles in their indecision zone, with a resulting reduction in safety. Similarly, long passage times tend to increase delay and cause phase termination by max-out (which also reduces safety).

This section examines the safety trade-offs associated with alternative detection designs. The insights obtained are used to develop a procedure for evaluating the safety of the detection design. The examination is based on the probability of stopping at the onset of the yellow indication as a function of a vehicle's travel time to the intersection. Research indicates that this probability increases with increasing travel time to the stop line (*3*). The complement of this probability is the probability that the driver will continue through the intersection (i.e., go). The probability of stopping and the probability of going are shown in Figure 3-1.

At a given travel time from the stop line, a "dilemma" is defined to exist if there is both a nonzero probability of stopping and a nonzero probability of going. The probability of a dilemma is represented as the product of these two probabilities. This probability is shown by the curve in Figure 3-2. The area under this curve represents the "possible exposure" to a dilemma on an intersection approach that does not have advance detection. The probability of a dilemma is at its maximum value when the probability of stopping and the probability of going both equal 0.5.

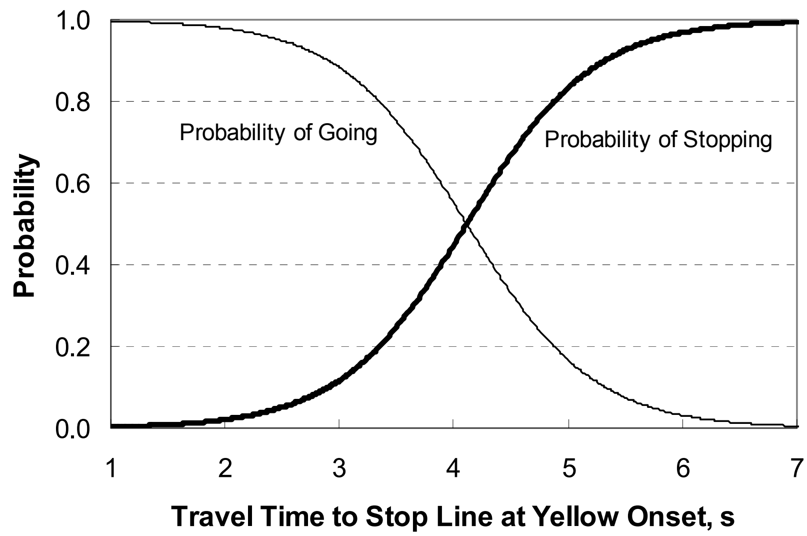


Figure 3-1. Probability of Stopping or Going at Yellow Onset.

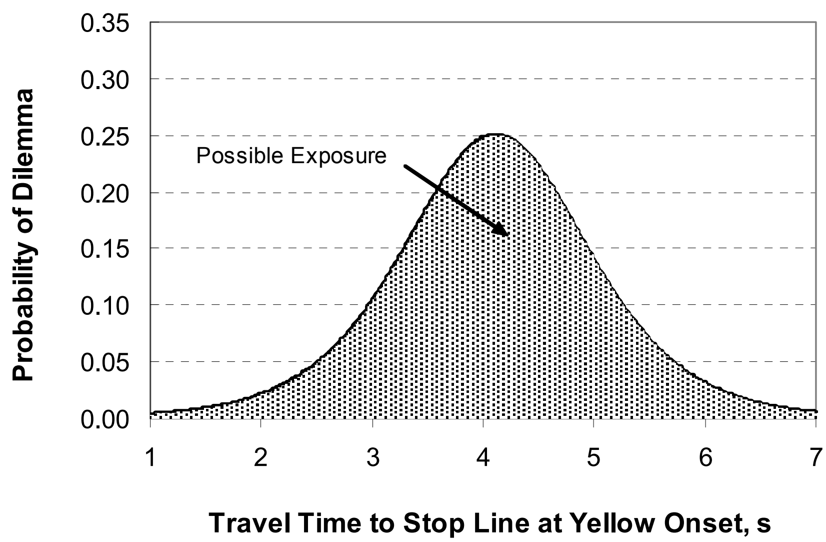


Figure 3-2. Probability of Dilemma.

Advance detection is used to minimize the exposure to the stop-or-go dilemma by sensing when a vehicle is in its indecision zone and extending the green indication until the vehicle is downstream of this zone. Ideally, the first advance detector would be placed 7 s travel time from the stop line and the passage time setting would be sufficiently long to hold the green until the vehicle reaches the next advance detector or the stop line.

In the aforementioned idealized situation, the driver's exposure to a dilemma is eliminated by the detection design. However, practical considerations limit the number of advance detectors that can be used as well as their maximum distance from the stop line. Moreover, experience indicates that full indecision zone coverage results in lengthy green extension during moderate to high volumes. Lengthy extension causes vehicles waiting for service in conflicting phases to incur significant delay and often results in phase termination by max out. When a phase terminates by max out, any indecision zone protection provided by the advance detection is lost. For these reasons, the detection design and passage time are often designed to provide indecision zone protection for only that portion of the traffic stream associated with the greatest probability of dilemma.

Figure 3-3 illustrates the manner by which advance detection can provide indecision zone protection for an intersection approach. The approach has three advance detectors and a passage time of about 1.0 s. The most distant advance detector is located at a travel time of 5.2 s. This particular detection design is not a "good" design because the passage time is inadequate to carry the vehicle to the next detector before passage time expires. Thus, the yellow will be presented while the vehicle is in its indecision zone. All such exposures to a dilemma on this approach are illustrated by the shaded portions under the curve. In contrast, the instances where indecision zone protection is provided are illustrated by the unshaded portions under the curve. The summation of the unshaded portions represents the "detector coverage."

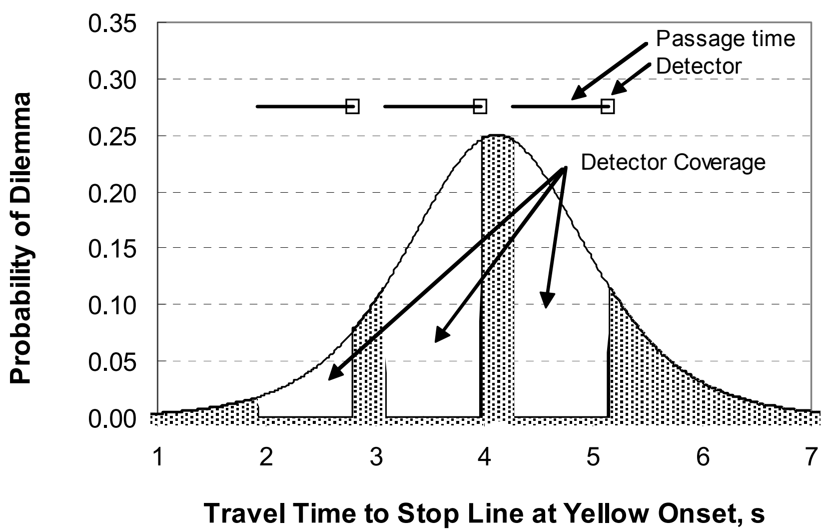


Figure 3-3. Indecision Zone Protection Provided by Advance Detection.

A useful measure of the quality of an advance detection design is the probability of indecision zone coverage. It represents the probability that a vehicle will receive indecision zone protection when traveling along the intersection approach at the onset of the yellow indication. It is computed as the ratio of detector coverage to possible exposure and is defined using the following equation:

$$P_{cov|gap} = \frac{Detector\ \ Coverage}{Possible\ \ Exposure} \quad \textbf{(5)}$$

where,

$P_{cov|gap}$ = probability of indecision zone coverage, given that the phase ends by gap out.

To accurately quantify detector coverage and possible exposure, the probability of dilemma must be assessed for the distribution of approach speeds. This assessment can be computed for both statistics in a discrete manner by evaluating every possible speed in 1-mph increments. Specifically, the overall detector coverage and possible exposure used in Equation 5 are each computed as a weighted average of their individual values for each speed, where the weight used for each individual value equals the probability of the corresponding speed.

When the green interval is extended to its maximum limit, the phase ends arbitrarily and any indecision zone protection provided by the detection design is lost. Thus, the probability of max-out must be included in the evaluation of detection coverage. The following equation is used to compute the overall probability of coverage, with consideration given to the probability of phase termination by max out.

$$P_{cov} = 1 - \left((1 - P_{cov|gap})(1 - P_{max}) + (1 - P_{cov|max})\, P_{max}\right) \tag{6}$$

where,

P_{cov} = probability of indecision zone coverage;

P_{max} = probability of max-out; and

$P_{cov|max}$ = probability of indecision zone coverage, given that the phase ends by max out (= 0.0).

As a final step, the probability of indecision zone coverage (i.e., detection coverage) is adjusted to address performance for the expected range of traffic volume. A volume-weighted average probability is computed for this purpose. This overall average is computed using the following equation:

$$P_{cov,tot} = 1 - \frac{\sum q_i (1 - P_{cov,i})}{\sum q_i} \tag{7}$$

where,

$P_{cov,\,tot}$ = weighted average probability of indecision zone coverage; and

q_i = volume for subject phase in scenario i, veh/h/lane.

The probability of indecision zone coverage can be used to assess the level of safety associated with alternative detection designs. A larger probability indicates greater indecision zone coverage and theoretically fewer crashes. A value of 1.0 indicates that max-out never occurs and the entire traffic speed distribution is provided detection coverage. A low probability indicates that the phase frequently ends by max-out, the advance loops are poorly located, the advance loops are too few in number, or the passage time is too short to allow vehicles to reach subsequent advance detectors in time to extend the green interval.

EVALUATION OF EXISTING DESIGNS

This part of the chapter describes an evaluation of the 8 Districts detection layout, as described in Table 3-1. The evaluation consisted of the application of the framework described in the previous part, but repeated for each of several analysis scenarios. Each scenario consists of a unique combination of design speed, 85th percentile speed, traffic volume, stop line detector operation, and passage time. The results of the evaluation were used to define application guidelines for the 8 Districts design. These guidelines identify a range of speeds and passage times for which each design will provide acceptable detection coverage.

Analysis Scenarios

All analysis scenarios were based on traffic and signalization conditions at an isolated intersection serving two intersecting highways. The intersection operated with a simple "two-phase" phase sequence where the major-road movements are served in one phase and the minor-road movements are served in the second phase. Turn movements were not represented at the intersection because they are not relevant to the evaluation of detection designs for through movements. Table 3-2 lists the intersection conditions that are the same for all analysis scenarios.

Table 3-2. Common Variables for Analysis Scenarios.

Category	Variable	Values by Road Category	
		Major Road	Minor Road
Signalization	Minimum green setting, s	10	10
	Maximum green setting,[1] s	40, 60, 80	40
	Yellow change plus red clearance interval, s (speeds listed are 85th percentile speed)	30 mph: 4.0 35 mph: 4.5 40 mph: 4.0 45 mph: 4.5 50 mph: 5.0 55 mph: 5.5 60 mph: 5.5 65 mph: 6.0 70 mph: 6.5 75 mph: 7.0	4.0
	Passage time, s	varies	1.6
	Phase recall	Recall to Minimum	none
	Detection mode	Presence	Presence
	Detector memory	Nonlocking	Nonlocking
Detection Layout	Stop line detector length, ft	40	40
	Advance detector location, ft	varies	not used
Other	Through lanes per approach	2	1
	Saturation flow rate, veh/h/lane	1700	1700
	Start up lost time, s	2.0	2.0

Note:
1 - Major-road maximum green varies with major-road volume as follows: 40, 40, 60, and 80 s for volumes of 100, 200, 300, and 400 veh/h/lane, respectively.

The variables used to define the analysis scenarios are listed in Table 3-3. Each unique combination of variables represents one analysis scenario. A total of 2016 scenarios (= 6×7×4×2×6) were created through this process. All scenarios include a 40-ft stop line detector. Efficient operation with this length of detector requires a minimum passage time of 1.0 s. Preliminary analyses indicated that the passage time providing the best operation was in the range of 1.0 to 2.0 s.

The 85th percentile speed was varied in 5 mph increments over a range of feasible values. This range was linked to the design speed of the detection layout, as indicated in Table 3-1.

Specifically, seven values of the 85th percentile speed were computed for each detection design speed. For example, 85th percentile speeds of 35, 40, 45, 50, 55, 60, and 65 mph were evaluated for the design speed of 50 mph. This approach allowed each design to be evaluated for a range of speeds. More robust designs would accommodate a wider range of speeds, such as may occur over the course of the day, month, or year, or when the speed limit is changed.

Table 3-3. Alternative Variable Values for Analysis Scenarios.

Variable	Values
Detection design speed, mph	45, 50, 44, 60, 65, 70
85th percentile speed, [1] mph	Design speed ± 0, 5, 10, 15
Major-road traffic volume, veh/h/lane	100, 200, 300, 400
Minor-road traffic volume, [2] veh/h/lane	40, 80, 120, 160
Stop line detector operation	Continuously Active, Deactivated
Passage time, s	1.0, 1.2, 1.4, 1.6, 1.8, 2.0

Notes:

1 - Seven combinations of 85th percentile speed. Combinations are computed to equal the design speed plus 0, 5, 10, or 15 mph and design speed minus 5, 10, or 15 mph.

2 - Minor-road volume is 40 percent of major-road volume for each scenario.

Detector Coverage

Concepts

The typical relationship between detector coverage and 85th percentile speed is shown in Figure 3-4. As noted previously, detector coverage is represented as the volume-weighted average probability of indecision zone coverage. The trend lines shown in the figure represent a detector layout based on a design speed of 55 mph and a stop line detector that is active. Each curve represents a different passage time value. The largest probability of coverage is shown to occur when the 85th percentile traffic speed is close to the design speed. However, the decrease in probability is relatively small when the 85th percentile speed is only slightly faster or slower than the design speed. A passage time of 1.0 s is shown to provide the best coverage for this detection design. The coverage is lower when the passage time is slightly shorter, or longer, than 1.0 s.

The trends in Figure 3-4 are typical of those found for the other detector designs. In each case, there was an optimal combination of speed and passage time. Fortunately, the coverage trend line is often sufficiently flat near these optimal values that slight variations from optimal do not significantly degrade the coverage. This characteristic suggests that the detection designs evaluated are robust such that they can accommodate changes in speed that may occur throughout the day, month, or year. In terms of the passage time, this characteristic allows for small adjustments to be made in the passage time setting for the purpose of reducing delay without significantly compromising the design's indecision zone coverage.

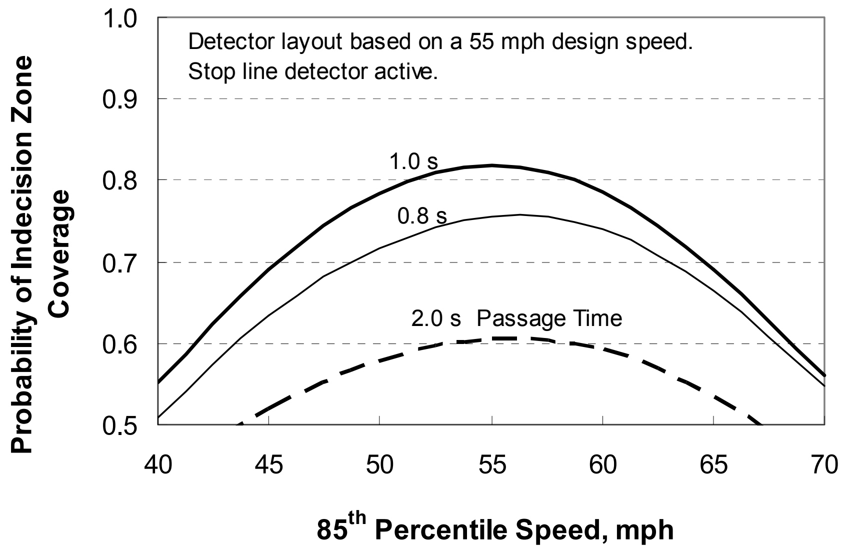


Figure 3-4. Typical Detector Coverage Relationships.

Guideline Development Process

Optimum values of speed and passage time were identified for each of the 8 District detection layouts using the scenarios described in the previous section. Relationships similar to those shown in Figure 3-4 were developed for each layout. The resulting detector coverage is shown in Figure 3-5 as a function of design speed and stop line detector operation.

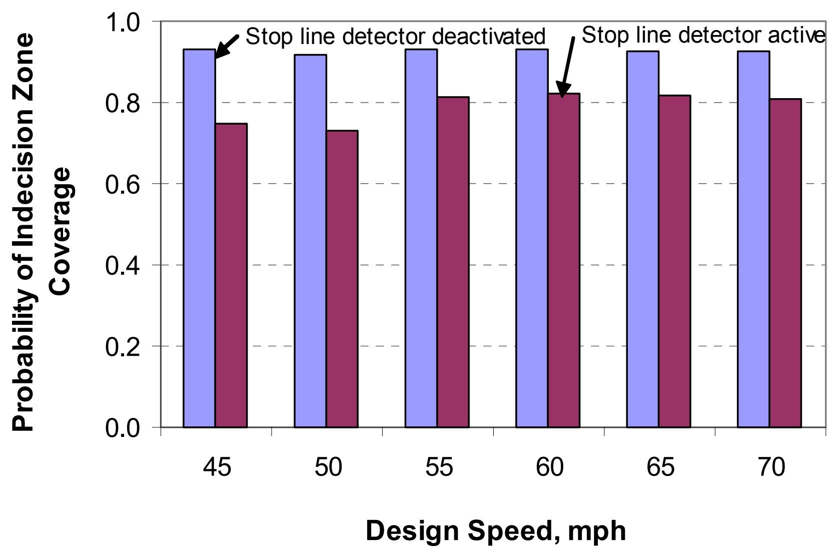


Figure 3-5. Detector Coverage as a Function of Design Speed and Detector Operation.

The trends shown in Figure 3-5 indicate that the coverage provided by each detection design is about the same, regardless of design speed. More notably, the stop line detector operation is shown to have a significant influence on coverage. Leaving the stop line detector active during the green interval consistently reduces the coverage to 80 percent or less. That is, under otherwise optimal passage time and speed conditions, an advance detection system with an active stop line detector is likely to provide indecision zone protection for only 70 to 80 percent of vehicles. In contrast, the deactivated stop line detector operation will provide detector coverage to about 93 percent of vehicles. These percentages shown in Figure 3-5 represent low-to-moderate volumes. They decrease by about 10 percent for high volume conditions (i.e., 600 veh/h/lane or more).

The passage time and speed combinations that provide optimum coverage for each of the 8 District detection layouts were used to identify alternative passage times that provided near-optimum coverage for a reasonable range in speed. These alternative times and speed ranges formed the basis for the application guidelines for the 8 District layouts. The following criteria were used to were used to develop these guidelines:

- For a given design speed, the minimum 85th percentile speed equals the optimum 85th percentile speed minus 5 mph and the maximum 85th percentile speed considered equals the optimum speed plus 5 mph.
- The probability of indecision zone coverage for the alternative passage time (when evaluated at its minimum and maximum 85th percentile speeds) has to be within 0.05 of the optimum coverage probability.

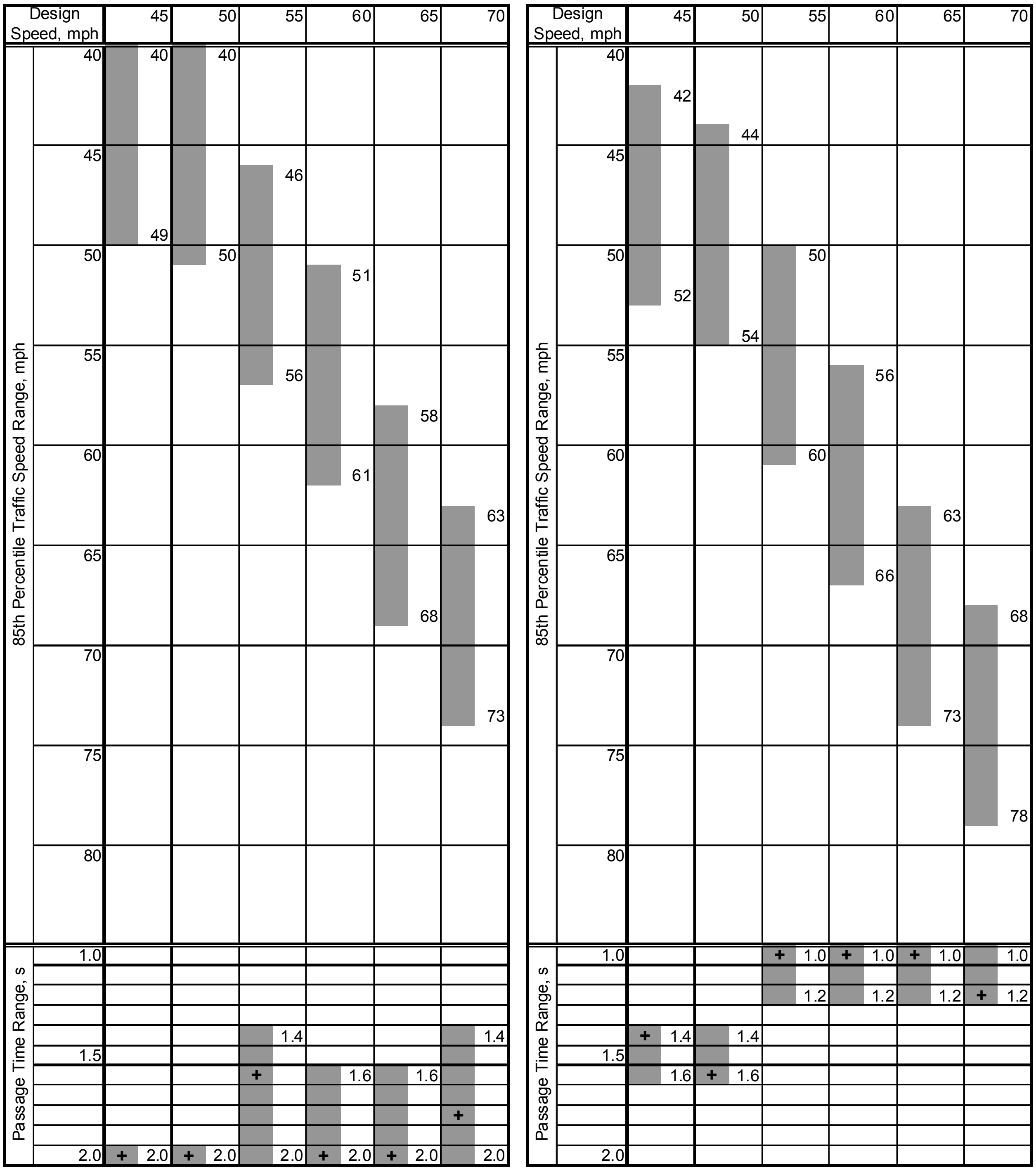
Using the aforementioned criteria, a range of acceptable passage times was identified for a 10-mph range centered on the optimum coverage speed. Combinations of passage times within this range would correspond to coverage probabilities that are within 0.05 of the optimum coverage.

Application Guidelines

The application guidelines based on detection coverage analysis are shown in Figure 3-6. Figure 3-6a applies when the stop line detector is deactivated. It is also applicable if stop line detection is not provided. Figure 3-6b is used when the stop line detector is continuously active. Each column in Figure 3-6a or 3-6b corresponds to one detection layout for a given design speed.

Figure 3-6 can be used to select the appropriate passage time for a given detection layout. For example, consider an intersection with a detection layout corresponding to a 60-mph design speed and a stop line detector that is deactivated following the first gap out. Figure 3-6a indicates that an effective passage time for this location is 1.6 to 2.0 s. The "+" symbol indicates that a passage time of 2.0 s provides the best overall coverage. In contrast, if the stop line detector is active, then Figure 3-6b indicates that the passage time should range from 1.0 to 1.2 s. The passage time of 1.0 s provides the best coverage; however, values as large as 1.2 s provide acceptable coverage.

Figure 3-6 can be used to select the appropriate detection layout for a given intersection approach. For example, consider an intersection approach with an 85th percentile traffic speed of 60 mph. The upper section of Figure 3-6a (labeled "85th Percentile Traffic Speed Range") indicates that the 8 District detection layout for a 60-mph design speed will provide effective coverage. In fact, this particular layout provides good coverage should the 85th percentile speed drop to 51 mph (or increase to 61 mph) during the day, month, or year.



"+" indicates desirable passage time.

a. Stop Line Detector Deactivated. **b. Stop Line Detector Active.**

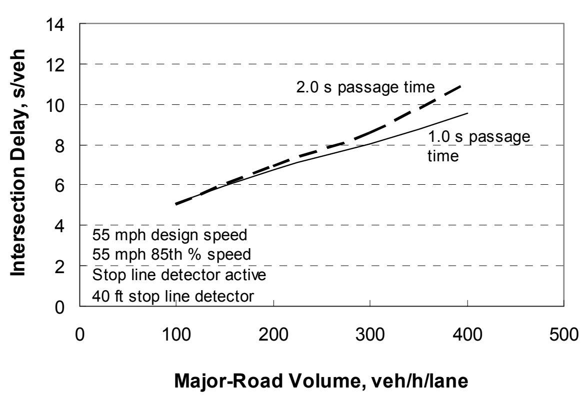
Figure 3-6. Effective Passage Time Settings by Speed and Detector Operation.

The traffic speed ranges shown in Figure 3-6a are similar to those shown in Figure 3-6b in most cases. However, the passage time ranges in Figure 3-6b are narrower than those in Figure 3-6a, and the passage time values are lower. These differences are a consequence of the stop line detector operation. When the stop line detector remains active, it continues to extend the green phase and increases the probability of max-out. To mitigate the increase in max-out probability, it becomes necessary to use shorter passage times. However, shorter passage times increase the probability of indecision zone exposure when gap-out occurs. Thus, a reduction in indecision zone coverage results when the stop line detector is allowed to remain active, as shown previously in Figure 3-5.

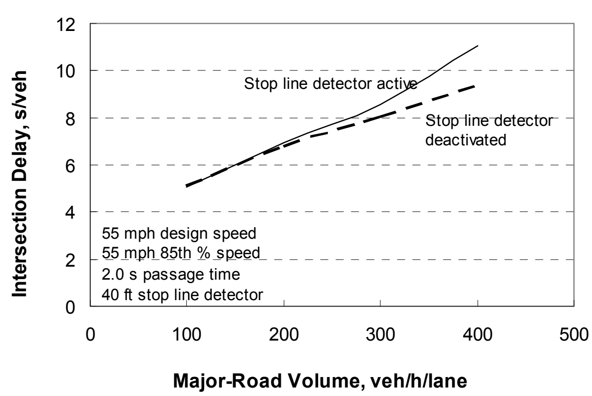
Control Delay and Max-Out Probability

The section examines the effect of passage time and stop line detector operation on intersection delay and the max-out probability for the major-road through movement. Intersection delay represents the total delay incurred for all intersecting movements divided by the total number of vehicles served by the intersection. As such, it provides an indication of the effect of a detection design on the overall intersection operation.

The effect of passage time and stop line detector operation on intersection delay is shown in Figure 3-7. In Figure 3-7a, it is shown that a 2.0 s passage time is associated with longer delay than a 1.0 s passage time. If fact, passage times shorter than 1.0 s are also associated with a longer delay. This latter trend occurs because the phase occasionally gaps out prematurely and results in previously queued vehicles having to wait for another signal cycle. In general, passage times of 1.0 to 1.5 s provide the lowest delay when a 40-ft detector is used at the stop line. These values increase if the stop line detector length is shorter than 40 ft.



a. Passage Time.

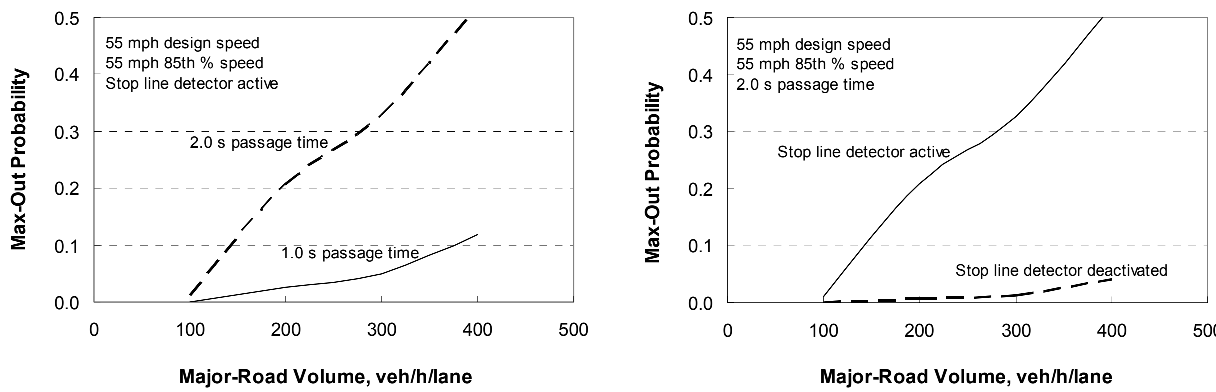


b. Stop Line Detector Operation.

Figure 3-7. Delay as a Function of Passage Time and Detector Operation.

The effect of stop line detector operation is shown in Figure 3-7b. The trend lines indicate that this operation has negligible effect on delay for a lane volume of 300 veh/h/lane or less. Using the deactivated operation will yield lower delay when the volume exceeds this level because the probability of max out will decrease.

The effect of passage time and stop line detector operation on max-out probability is shown in Figure 3-8. As indicated by Figure 3-8a, when passage time is increased, the major-road green interval is extended to its maximum more frequently, especially at higher volumes. As a result, minor-road vehicles arriving at the intersection during the red indication are forced to wait longer for service. Phase termination by max-out also has a safety consequence because it effectively disables the indecision zone protection provided by the advance detectors. Figure 3-8b indicates that the deactivated stop line detector operation significantly reduces max-out probability.



a. Passage Time. **b. Stop Line Detector Operation.**

Figure 3-8. Max-Out Probability as a Function of Passage Time and Detector Operation.

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CHAPTER 4. EVALUATION OF SIGNAL CONTROLLER SETTINGS

OVERVIEW

Interviews with TxDOT engineers and technicians were conducted to identify areas where additional signal timing guidance may be helpful. The findings from these interviews indicated the existence of variation among districts in the preferred maximum and minimum green settings for actuated phases. This chapter describes the research undertaken to develop recommended guidelines for selecting appropriate maximum and minimum green settings. The first part of the chapter summarizes a review of practice on this topic. The last part summarizes an evaluation of alternative maximum and minimum green settings.

REVIEW OF PRACTICE

This part of the chapter summarizes a review of the literature on the topic of maximum green setting and minimum green setting for an actuated phase. The review includes the findings from several surveys of practice and the guidance provided in several engineering reference documents. The first section addresses the maximum green setting and the second section addresses the minimum green setting.

Maximum Green

This section describes the maximum green settings used in practice. The discussion addresses the setting values used for both left-turn phases and through phases. The first subsection presents the findings from several surveys on this topic. The second subsection presents the guidance offered in various documents for determining the appropriate maximum green setting.

Survey of Engineers

This subsection summarizes the findings from two surveys that were conducted on topics related to signal timing practice. One survey was conducted by the researchers. It focused on the practices of TxDOT engineers. The other survey was conducted by Tarnoff and Ordonez (*1*). They surveyed more than 100 state, city, and county agencies responsible for traffic signal operations. One question in this survey inquired about the typical maximum "phase" time used in the jurisdiction, as opposed to the maximum green setting. However, the context of the discussion indicates that the responses provided to the survey refer to the maximum green setting. No distinction was made between values used for left-turn phases and for through phases. The distribution of the replies to this question is shown in Figure 4-1. It is labeled "Nationwide Survey."

The trends in Figure 4-1 show that a wide range of maximum green settings is used nationwide. The nationwide survey indicates that most agencies (37 percent) use maximum green settings in the range of 51 to 100 s. In contrast, only 5 percent of agencies use maximum green settings less than 50 s. Although not explicitly stated, it is likely that the respondents reported the value they use for a through-movement phase (as opposed to a left-turn phase).

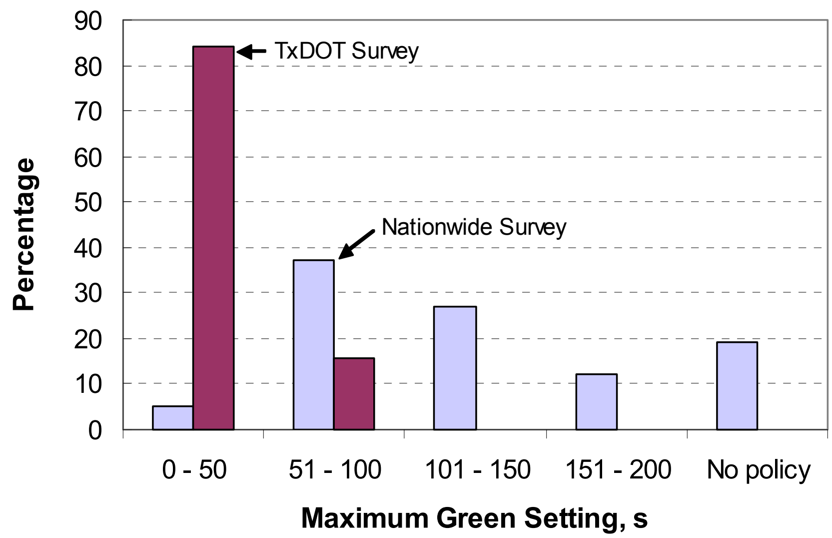

Figure 4-1. Distribution of Maximum Green Values Used in Practice.

Also shown in Figure 4-1 are the maximum green settings reported during the survey of TxDOT districts. These findings are based on the responses provided by 19 of the 25 districts. No response was provided to this question by six districts. The question was specific to the maximum green used for the typical through movement phase. Almost all districts (84 percent) use a maximum value of 45 s or less. The average of the reported values is 38 s with a range of 20 to 60 s.

Further examination of the TxDOT responses indicates that there was some correlation between the maximum green setting for through phases and road usage. The districts with the most vehicle-miles of travel typically used maximum green settings in the range of 40 to 60 s. In contrast, those districts with the least vehicle-miles of travel typically used maximum green settings in the range of 25 to 40 s.

The TxDOT districts were also asked about the typical maximum green setting used for left-turn phases. The average maximum green setting for this case is 20 s with a range of 10 to 35 s. A comparison of the maximum green settings reported by each district indicates that it is fairly common to have the maximum green for a left-turn phase equal about one-half that used for the through phase.

Literature Review

Maximum Green Calculation. A review of the literature indicates that guidance regarding the maximum green setting is predominantly based on traffic volume, where the volume considered is that of the peak period (*2*). The stated objective is to choose a maximum green setting that is sufficiently large that the frequency of extension to the maximum green limit is minimal during the day and, if it occurs, it is limited to the peak demand periods.

Two basic approaches are followed when using traffic volume to estimate the maximum green setting. One approach is based on the calculation of the optimum green duration G_o for the intersection based on pretimed signal operation (*3, 4, 5, 6*). This duration can be obtained from a software product (e.g., PASSER II) that finds the minimum-delay timing plan through some type of optimization algorithm. It can also be calculated using the critical movement analysis technique (*6*). With this procedure, the following equation is used to estimate the minimum-delay cycle length:

$$C_o = \frac{1.5\,L + 5}{1 - (\sum v_{c,i}) / 1600} \qquad \textbf{(8)}$$

where,

C_o = minimum-delay cycle length, s;
L = total lost time (= 4 n_p), s;
n_p = largest number of phases used in a ring (consider both rings); and
$v_{c,i}$ = critical movement volume for phase *i*, veh/h/lane.

The optimum green duration is computed as:

$$G_{o,i} = \frac{v_{c,i}}{\sum v_{c,i}} (C_o - 4 n_p) + 4 - Y_i \qquad \textbf{(9)}$$

where,

G_o = optimum green interval duration for phase *i*, s;
L = total lost time (= 4 n_p), s; and
Y_i = change period for phase *i*, s.

The maximum green interval is computed using the following equation:

$$G_{\max,i} = f_1\ G_{o,i} \qquad \textbf{(10)}$$

where,

$G_{max,\,i}$ = maximum green setting phase *i*, s; and
f_1 = adjustment factor.

The recommended adjustment factor typically varies from 1.25 to 1.50 (*3, 4, 5, 6*). Lin (*4*) indicates that intersection operation is not significantly affected by the value chosen for the factor, provided that it is used to establish the maximum green setting for all signal phases. However, larger values of this factor should be used cautiously because long maximum green settings can yield lengthy delays when a detector fails or a car stalls on the detector (*2*).

A second approach for determining the maximum green setting is based on the average queue service time during the peak period (*2, 7*). Initially, a desired cycle length for equivalent pretimed operation is selected. Then, the average number of arrivals per cycle *n* is computed. Finally, the queue service time is computed using the following equation:

$$G_{s,i} = 3 + 2\,n_i \qquad \textbf{(11)}$$

where,

G_s = average queue service time for phase i, s; and
n_i = number of arrivals per cycle for phase i, veh/cycle.

The maximum green interval is computed using the following equation:

$$G_{\max,i} = f_2 \; G_{s,i} \tag{12}$$

where,

$G_{max,\,i}$ = maximum green setting phase i, s; and
f_2 = adjustment factor.

The recommended adjustment factor varies from 1.2 to 1.3 and is intended to estimate the queue service time needed for most signal cycles. The maximum green values obtained from this approach are typically about 25 percent smaller than those obtained from Equation 10.

Typical Maximum Green Settings. Two state agencies have specified typical maximum green settings in their respective signal timing manuals. These settings are summarized in Table 4-1. The range of maximum green settings offered in this table for the major approach through phase is consistent with the findings noted in Figure 4-1 for the nationwide survey. Also, the range offered for the left-turn phase is about one-half that use for the through phase. This trend is consistent with the findings from the survey of TxDOT districts.

Table 4-1. Typical Ranges of Maximum Green Settings Used by Two State DOTs.

Phase	Condition	Maximum Green Setting, s	State DOT
Left-turn	All	10 to 45	Minnesota (*6*)
Through	Minor approach	20 to 75	
	Major approach	30 to 120	
Left-turn or through	All	15 to 60	Utah (*5*)

Minimum Green

This section describes the minimum green settings used in practice. The discussion addresses the setting values used for both left-turn phases and through phases. The first subsection presents the findings from two surveys on this topic. The second subsection presents the guidance offered in various documents for determining the appropriate minimum green setting.

Survey of Engineers

This subsection summarizes the findings from the two surveys described in the previous section. In the nationwide survey, one question inquired about the typical minimum "phase" time used in the jurisdiction, as opposed to the minimum green setting. However, the context of the discussion indicates that the responses provided to the survey refer to the minimum green setting.

No distinction was made between values used for left-turn phases and for through phases. The distribution of replies to this question is shown in Figure 4-2 and labeled as "Nationwide Survey."

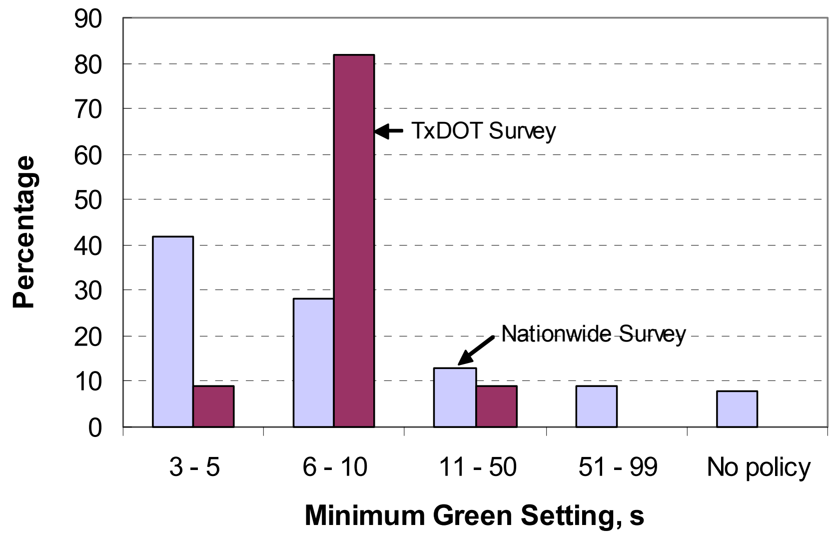

Figure 4-2. Distribution of Minimum Green Values Used in Practice.

The trends in Figure 4-2 show that a wide range of minimum green settings is used nationwide. Most agencies (42 percent) use minimum green settings in the range of 3 to 5 s. In contrast, only 9 percent of agencies use minimum green settings of 50 s or more. Although not explicitly stated, it is likely that the respondents reported the values used for through phases (as opposed to left-turn phases).

Also shown in Figure 4-2 are the minimum green settings reported during the survey of TxDOT districts. These findings are based on the responses provided by 22 of the 25 districts. No response was provided to this question by three districts. The question was specific to the minimum green used for the typical major-road through movement phase. Almost all districts (82 percent) use a minimum value in the range of 6 to 10 s. The average of the reported values is 9 s with a range of 5 to 20 s.

The TxDOT districts were also asked about the typical minimum green setting used for minor-road through phases and left-turn phases. The average minimum green setting for minor-road phases is 7 s with a range of 4 to 10 s. The average minimum green setting for a left-turn phase is 6 s, with a range of 4 to 10 s.

Literature Review

The review of the literature indicated that the minimum green setting for through movements may be based on one or more of the following considerations: driver expectancy, queue clearance time, and pedestrian crossing time. If two or more of these considerations are applicable, then the one requiring the longest minimum green value should be used to establish the minimum green setting.

Minimum Green Based on Driver Expectancy. Research indicates that drivers expect the signal indication to remain green for at least 3 to 5 s. Intervals that are shorter than 3 s are contrary to driver expectancy and tend to cause unsafe maneuvers (*2*).

One agency guideline indicates that the minimum green duration should be roughly equal to the time required by the first queued driver to react to the onset of the green indication and to travel well into, or through, the intersection (*7*). Similar guidance was reported by Tarnoff and Ordonez (*1*) in their survey of agency practice. Interviews with TxDOT staff indicate that the minimum green setting may need to be increased beyond 5 s when the number of trucks is significant because these vehicles require more time to start up and travel into the intersection.

Table 4-2 summarizes the guidance offered by several agencies related to the minimum green setting. As shown in the table, the minimum green setting for left-turn phase ranges from 4 to 7 s. This range is consistent with that identified in the survey of TxDOT practice.

Table 4-2. Typical Ranges of Minimum Green Settings Based on Driver Expectancy.

Phase	Condition	Approach Speed, mph	Minimum Green Setting (G_e), s	State DOT
Left-turn	All	any	4	South Dakota (*8*)
			4 to 5	Utah (*5*)
	Protected-permissive		5	Minnesota (*6*)
	Protected-only		7	Minnesota (*6*)
Through	Minor approach		7	South Dakota (*8*)
			4 to 5	Utah (*5*)
			10	Minnesota (*6*)
	Major approach		12	South Dakota (*8*)
		less than 45	15	Minnesota (*6*), Utah (*5*)
		45 or more	20	Minnesota (*6*), Utah (*5*)
Left-turn or through	Normal width intersection	any	4	Idaho (*7*)
	Wide intersection		5 to 6	Idaho (*7*)

Table 4-2 indicates that the minimum green setting for through phases varies widely among the referenced agencies. The range of settings is 4 to 20 s, with shorter values used for minor-road approaches to the intersection and longer values used for major-road approaches. The longest minimum green of 20 s is reserved for major roads with an approach speed of 45 mph or more. This range is consistent with that identified in the survey of TxDOT practice.

Minimum Green Based on Queue Clearance Time. The minimum green for queue clearance is needed when there are one or more advance detectors but no stop line detection. In this situation, the green indication will need to be presented for sufficient time to serve all vehicles that may be stopped between the nearest advance detector and the stop line. The following equation can be used for this purpose:

$$G_{q,i} = l_1 + h\,n_d \tag{13}$$

where,

$G_{q,i}$ = minimum green based on queue service time for phase *i*, s;
l_1 = start-up lost time, s;
n_d = maximum number of vehicles stored between detector and stop line (= D_d /25), veh; and
h = average saturation headway for a discharging queue, s/veh.

The use of Equation 13 to compute the minimum green setting is described in two agency documents (*6, 8*). Specifically, Minnesota DOT guidelines recommend a lost time of 3 s and an average headway of 2.0 s/veh (*6*). South Dakota DOT guidelines do not specify values for lost time and average headway (*8*). However, their recommended minimum green values can be computed using Equation 13 with a lost time of 7.7 s and an average headway of 1.2 s/veh.

Minimum Green Based on Pedestrian Crossing Time. The minimum green setting must satisfy pedestrian crossing needs for those through phases that are not associated with a pedestrian push button and for which a pedestrian demand is known to exist. Guidance offered by four agencies and the *Texas MUTCD* (*9*) is consistent in their recommendation of the following equation for computing the minimum green needed to serve pedestrians (*2, 5, 6, 8*).

$$G_{p,i} = P_{rt} + \frac{D_c}{v_p} - (Y + R_c) \tag{14}$$

where,

$G_{p,i}$ = minimum green based on pedestrian crossing time for phase *i*, s;
P_{rt} = pedestrian perception-reaction and curb departure time, s;
D_c = pedestrian crossing distance, ft;
v_p = pedestrian walking speed, ft/s;
Y = yellow change interval, s; and
R_c = red clearance interval, s.

A pedestrian perception-reaction time of 4.0 s is recommended by all four agencies. However, one agency suggests that 7 s is appropriate for moderate pedestrian volume (i.e., 10 to 20 ped/cycle) and even longer values can be used for heavy pedestrian volumes. The crossing distance is specified by the *Texas MUTCD* (*9*) as a curb-to-curb distance, as measured along the crosswalk. A majority of TxDOT districts do not include the yellow change and red clearance interval terms in Equation 14 when computing the minimum green setting.

The *Texas MUTCD* (*9*) recommends a walking speed of 4.0 ft/sec. However, the *Pedestrian Facilities User Guide* (*11*) recommends a maximum walking speed of 3.5 ft/s. This guide also suggests that a slower walking speed should be used in areas where there is a heavy concentration of elderly pedestrians or children.

Additional Guidance on Minimum Green. Other minimum green guidance was found in the literature that extended beyond considerations of driver expectancy, queue clearance time, and pedestrian crossing time. Specifically, the Utah DOT document advises that the minimum green setting should equal the average queue service time (as obtained from Equation 11) when either of the following two conditions exists (*5*):

- detection is not provided for a movement (e.g., it is served by the non-actuated phase at a semi-actuated intersection), or
- detection quality has degraded to an unreliable condition.

EVALUATION OF ALTERNATIVE SETTINGS

This part of the chapter consists of two sections. The first section describes the procedure used to evaluate alternative maximum and minimum green settings. The second section summarizes the findings from the evaluation.

Evaluation Process

The evaluation process focused on an examination of the effect of alternative minimum and maximum green settings on control delay. Control delay was computed using the methodology described in Chapter 16 of the *Highway Capacity Manual* (*10*).

The evaluation of maximum green setting focused on the relationship between the adjustment factor in Equation 10 and average intersection delay. The objective was to determine the factor value that yielded the lowest intersection delay. The analysis considered a range of factor values and a range of major- and minor-road approach volumes. Equations 8 and 9 were used to estimate the optimal green interval duration.

The evaluation of minimum green setting focused on the relationship between minimum green duration, major-road through movement volume, total minor-movement volume, and intersection delay. The objective was to determine the minimum green value that yielded the lowest intersection delay. The total minor-movement volume represents the sum of the volumes for the minor-road approaches and the left-turn movements from the major road.

Findings

Maximum Green Setting

This section describes the findings from the evaluation of alternative maximum and minimum green settings. Figure 4-3 illustrates the relationship between intersection delay and the maximum green adjustment factor. Three trend lines are shown in this figure and represent different volume levels, ranging from moderate to high levels. The trend in the lines indicates that intersection delay is relatively insensitive to the adjustment factor value. This trend is consistent with the findings reported by Lin (*4*). Although not shown, intersection delay was observed to increase rapidly as the factor value decreased below 1.0. It also increased noticeably when the factor value increased above 1.5.

The maximum green setting for through phases based on delay considerations is listed in Table 4-3. The values listed in this table were computed using Equations 8, 9, and 10 with an adjustment factor of 1.3. The minor approach volume was assumed to equal one-half that of the major approach. The maximum green settings for the major-major configuration with 700 veh/h/lane are quite large and illustrate the sensitivity of Equation 8 to near-capacity conditions.

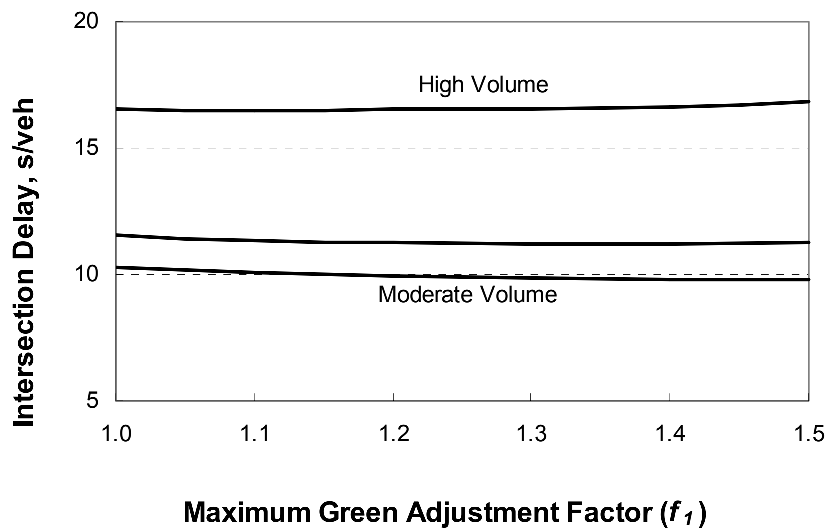

Figure 4-3. Variation in Delay as a Function of Maximum Green.

Table 4-3. Computed Maximum Green Setting for Through Phases.

Intersection Configuration	Critical Phase Count [1]	Through Phase on...	Maximum Green Setting, s by Major Approach Volume, veh/h/lane [2]				
			300	400	500	600	700
Major - Minor	2	Major approach	15	20	25	40	70
		Minor approach	10	10	15	20	35
	3	Major approach	15	20	25	35	50
		Minor approach	10	10	15	20	25
Major - Major	3	Both approaches	15	20	35	60	200
	4	Both approaches	20	25	40	65	140

Notes:
1 - Critical number of phases for the intersection:
"2": Only through phases are provided. No left-turn phases are provided.
"3": Through phases plus left-turn phases for one major road.
"4": Through phases plus left-turn phases for both major roads.
2 - The major-road approach volume used in the table represents the larger volume of the two major-road approaches. For the major-major intersection, this volume is estimated as the average of the major approach volume for both intersecting roads.

To illustrate the interpretation of Table 4-3, consider the intersection of two major roads. The intersection has three critical phases. One road has a northbound approach volume of 400 veh/h/lane and a southbound approach volume of 650 veh/h/lane. The approach volume for the southbound road equals the larger of the two approach volumes (i.e., 650 veh/h/lane). The approach volume for the intersecting road is determined in a similar manner and found to equal 550 veh/h/lane. Footnote 2 of Table 4-3 indicates that the average of these two major approach volumes (i.e., 600 veh/h/lane) should be used in the table to estimate the maximum green setting. Based on the information provided, a maximum green setting of 60 s is appropriate for the through phases on each of the intersecting roads.

Minimum Green Setting

The findings from the examination of minimum green duration are shown in Figure 4-4. The trend line in this figure applies when the total minor-movement volume is in the range of about 100 to 200 veh/h. When this volume is below or above the stated range, the slope of the line decreases. For significant deviations from this range, the trend line is nearly flat and varies from 5 to 10 s.

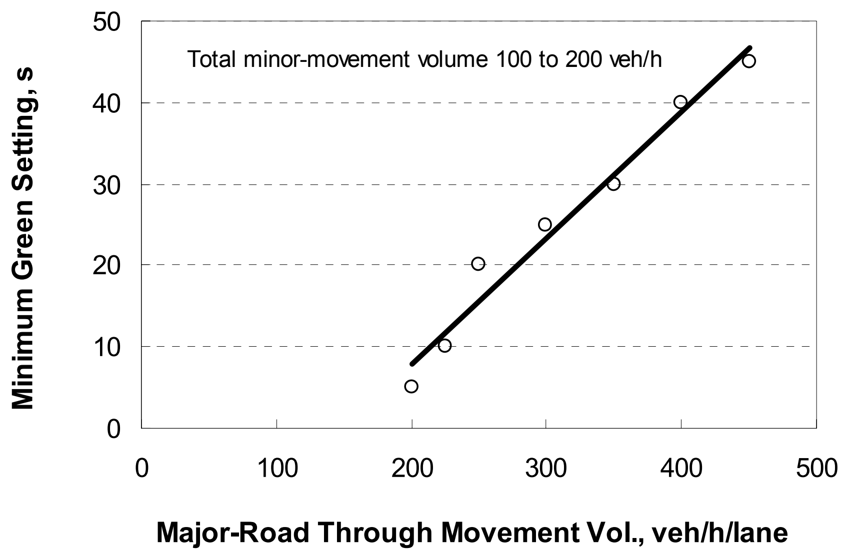

Figure 4-4. Optimum Minimum Green Setting When Total Minor-Movement Volume is between 100 and 200 veh/hr.

The trend line in Figure 4-4 indicates the minimum green setting that yields the lowest total intersection delay for a given major-road through movement volume. This delay increases for other values of the minimum green setting. The reason for this trend is explained in the following paragraph.

Under actuated control, a phase is called when a vehicle travels across the assigned detector. When the minor-movement volume is light (i.e., about 150 veh/h total for all movements), calls for the phase serving minor-road vehicles tend to arrive with an average headway of 18 to 30 s. These calls arrive sufficiently far apart that they are individually served by the controller and at a time when the major-road approach queue has just cleared from the previous call. Higher minor-movement volumes arrive more frequently and are served in groups, so they do not increase the delay to the major-road movements. Lower minor-movement volumes produce longer call headways and, thereby, allow the major-road movement to retain the green for longer periods.

In summary, total minor-movement volume in the range of 100 to 200 veh/h can create a short cycle length and inefficient operation (i.e., one minor-movement vehicle served per cycle) when the major-road through movement has a minimum green of 5 to 10 s. For this volume range, total intersection delay can be reduced by using the minimum green setting obtained from Figure 4-4 for the major-road through phase.

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CHAPTER 5. DEVELOPMENT OF SIGNAL COORDINATION SOFTWARE

OVERVIEW

A wide array of signal timing software tools, such as PASSER II, TRANSYT-7F, and Synchro, are available for the purpose of evaluating and optimizing coordinated signal timing plans. These tools are quite powerful in their ability to quantify the effect of a wide variety of factors on signal system performance and traffic flow efficiency. Their power stems partly from the extensive detail to which the signal system is described in the form of input data and the complexity of their modeling algorithm. Hence, the power comes at a price. These tools require a large amount of input data and they require a significant time investment to learn and apply.

Transportation agencies are continually challenged with the need to update and maintain the quality of signal timing at the intersections in their jurisdiction. Traffic demand patterns change on a seasonal and an annual basis. Signal control equipment ages and is upgraded. Detectors wear out or fail and are replaced. As a result, signal timing plans are constantly in need of refinement. Unfortunately, agency responsibilities are broad and signal timing does not get updated as often as desired. One reason for this trend is that the development of optimal signal timing plans and their implementation takes a significant investment of time and resource by the agency.

In recognition of the aforementioned challenges, engineers have developed cost-effective techniques and rules-of-thumb to guide them in the development of effective signal timing plans. For example, engineers know that most non-coordinated signal phases operate in a fairly predictable range of phase splits. They also know that there are one or two cycle lengths that work well for a given arterial and associated intersection spacing. Experience tells them that a specific range of offsets will work for a given intersection. In many cases, this experience is sufficient to estimate a workable cycle length, offset, and phase splits for each intersection on the arterial street.

This chapter documents the development of a simple software tool that takes advantage of the aforementioned experiences, techniques, and rules that engineers apply when developing signal timing plans with limited resources. This tool can be used to develop a coordinated signal timing plan for an arterial street, where the plan is defined by the following four signal settings:

- cycle length,
- phase splits,
- offsets from system reference time, and
- left-turn phase sequence.

This software tool is referred to herein as the Texas Signal Coordination Optimizer (TSCO).

The signal settings identified in the previous paragraph are the basic descriptors of a signal timing plan for a given arterial street. The algorithm used in TSCO to optimize these settings is referred to as the "bandwidth technique." This technique does not require traffic data as a direct input and can be used to provide a reasonable timing plan when resources are not available for extensive traffic data collection and analysis. When these resources are available, the more powerful software tools (e.g., PASSER II) should be used because they are likely to produce more efficient timing plans.

This chapter is comprised of two parts. The first part describes the bandwidth technique. The last part describes the research conducted to validate the bandwidth technique implemented in TSCO. Guidelines for using TSCO are provided in the appendix.

BANDWIDTH TECHNIQUE DEVELOPMENT

This part of the chapter describes the research conducted to develop and validate the TSCO bandwidth technique. It consists of four sections. The first section describes the TSCO input data requirements. The second section describes the progression bandwidth and interference concept. The third section describes the half-integer offset optimization technique. The last section reviews various performance measures used to describe the quality of a coordinated signal timing plan.

Input Data Requirements

The bandwidth technique models the arterial street as a series of segments separated by nodes. A node can represent the location of a signalized intersection, a change in traffic speed, or both. The following data are used to describe the subject street:

- location of nodes along the street,
- node type (i.e., signalized intersection or speed change),
- segment speed,
- system cycle length,
- phase split (in percentage of system cycle length) for the coordinated street phases,
- change period,
- phase sequence for the coordinated street left-turn phases, and
- offset to coordinated phase 2 (optional).

Offset data are optional. The bandwidth technique can be used to evaluate a set of input offsets or search to find the optimal set of offsets.

Progression Bandwidth and Interference

This bandwidth technique is illustrated in Figure 5-1 for a street with four coordinated signals. On this example street there are four signalized intersections spaced evenly at a distance *L*. The dashed horizontal line associated with each intersection indicates the signal indication for each of two through movement phases (i.e., phase 2 and phase 6) that time concurrently. The solid portion of each line represents a red signal indication. The space between red indications shows when a green indication is displayed to the two through phases. Three signal cycles are shown. The signal at each intersection has a green interval equal to one-half the system cycle length *C*.

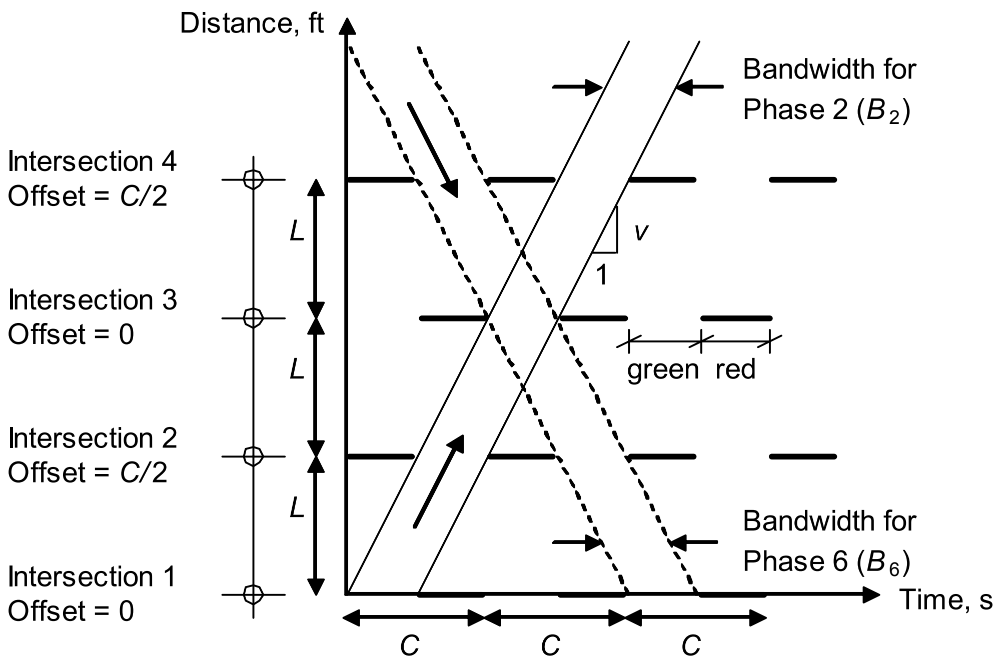


Figure 5-1. Illustration of Bandwidth Concept.

Progression bands are drawn for both through movement phases. Each band is defined by two parallel lines that have a slope equal to the progression speed v. The first line represents the trajectory of the first vehicle in the platoon as it travels down the street. The second line represents the last vehicle in the platoon. A vehicle will be able to pass through all four intersections without being stopped, if it travels within the band. Bandwidth is defined as the time duration of each band, as measured horizontally between the two trajectory lines.

The coordinated signal system shown in Figure 5-1 represents an idealized scenario where the green interval at each intersection is fully used for progressed movement. This type of progression is called "alternate ideal" progression because of the offset at adjacent intersections alternates between 0 and $C/2$ s (*1*). Whenever the odd-numbered intersections display a green indication for the coordinated movement, the even-numbered intersections are displaying a red indication, and vice versa. The following relationship between intersection spacing L, cycle length C, and progression speed v holds when alternate ideal progression is available for a signal system.

$$C = 2 \frac{L}{1.47\ v} \tag{15}$$

where,

C = cycle length, s;
L = segment length, ft; and
v = progression speed, mph.

Alternate ideal progression is not available for many coordinated signal systems because they do not have intersections uniformly spaced at a distance L. Signal timing plans for non-uniform signal spacing will be less efficient than those based on uniform spacing. In general, one or more of the intersections will be located such that it will tend to reduce, or interfere with, the band. This type of interference is illustrated in Figure 5-2 for intersections 2 and 3. Specifically, intersection 2 produces lower interference, clipping the leading edge of the band, while intersection 3 produces upper interference, clipping the trailing edge of the band.

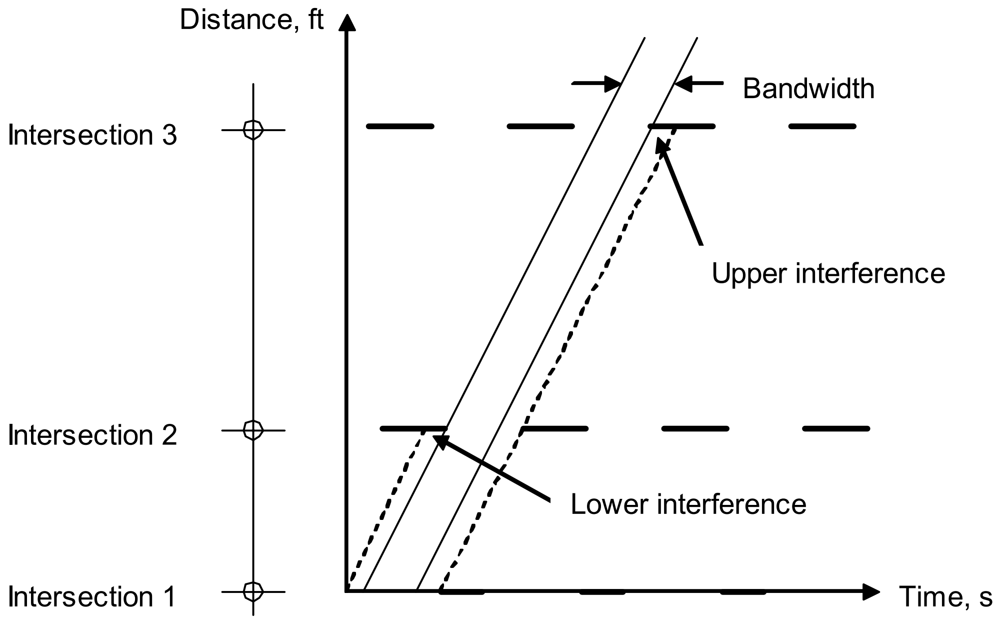


Figure 5-2. Upper and Lower Bandwidth Interference.

Lower interference at each intersection is computed by the following equation:

$$I_{L,i} = \mathrm{mod}[(\theta_i - \theta_c - T_{i,c}),\ C] \qquad \textbf{(16)}$$

where,

$I_{L,i}$ = lower interference at intersection i, s;
θ_i = offset at intersection i, s;
θ_c = offset at the critical intersection, s; and
$T_{i,c}$ = travel time between intersection i and the critical intersection, s.

The offset at intersection i is defined as the time difference between the start of green at this intersection and the system reference time zero. The critical intersection is defined as the intersection with the shortest green interval. This green interval will always constrain the bandwidth and, for this reason, is referred to as the critical green interval. In general, a critical green interval exists for each travel direction in a two-way progression timing plan. If the offsets at the non-critical intersections are chosen such that no interference is produced, the bandwidth will be equal to the duration of the critical green interval.

Upper interference at each intersection is computed by the following equation:

$$I_{U,i} = \text{mod}[(G_c - G_i - I_L),\ C] \tag{17}$$

where,

$I_{U,i}$ = upper interference at intersection i, s;
G_i = green duration at intersection i, s; and
G_c = green duration at the critical intersection, s.

Equations 16 and 17 can produce negative values. A negative value indicates that the intersection has slack green time (i.e., green time that cannot be used for progression). For example, if $T_{i,c}$ is equal to the difference in offsets between the two intersections (θ_i - θ_c), intersection i will produce 0 s of lower interference and $G_c - G_i$ s of upper interference. The resulting negative quantity $G_c - G_i$ represents unused green time at intersection i after the progressed platoon has passed. Slack time can be desirable if it occurs at the beginning of the green interval (i.e., negative lower interference) because it allows initial queues to clear before the progressed platoon arrives.

Half-Integer Offset Optimization

The offset optimization algorithm in TSCO is based on the concept of half-integer offset selection and bandwidth maximization (*2*). These concepts are briefly described in the following paragraphs.

Early signal coordination optimization methods were based on graphical representations of the half-integer concept (*3, 4*). This concept is based on the assumption that the optimal, two-way progression offset between two intersections is obtained when each intersection has an offset equal to one-half the duration of its green or red interval. The decision to reference the green or the red interval at a given intersection is based on consideration of both options and the selection of that option yielding the least interference.

Little (*2*) demonstrated that, in systems with optimally spaced signals, selecting half-integer offsets at every intersection will yield the maximum attainable bandwidth. Based on this principle, he developed an algorithm that optimized coordination by comparing the bandwidths obtained from all combinations of offset type at the system's intersections. This method can be extended to any signal system, even if the signals are not optimally spaced.

A variation of Little's algorithm has been used by several researchers to maximize bandwidth indirectly by using half-integer offsets to minimize interference (*5, 6, 7, 8, 9*). Using this variation, it is necessary to adjust the offsets of only the non-critical intersections, thus limiting the number of offset combinations that need to be evaluated. In contrast, Little's algorithm requires the explicit evaluation of all offset combinations. This variation is also used in TSCO.

Measures of Effectiveness

Several measures of effectiveness are used to describe the quality of a coordinated timing plan. These measures include total bandwidth, efficiency, and attainability. Total bandwidth is defined as follows:

$$B = B_2 + B_6 \tag{18}$$

where,

B = total bandwidth, s;
B_2 = phase 2 bandwidth, s; and
B_6 = phase 6 bandwidth, s.

The goal when developing a coordinated signal timing plan is to maximize total bandwidth, while leaving sufficient time in the cycle to serve the non-coordinated phases. Chaudhary et al. suggest that an effective signal timing plan should provide a total bandwidth of at least 15 s (*10*).

It is often possible to increase bandwidth by increasing the cycle length and through phase green intervals proportionally. However, excessively large cycle lengths often cause increased delay to traffic served by non-coordinated phases and may not yield offsetting improvements in bandwidth. For this reason, efficiency is a more appropriate measure of progression quality than bandwidth. Efficiency considers both bandwidth and cycle length. Specifically, it represents the portion of the cycle that is available for progression. It is defined as follows:

$$E = 100 \frac{B}{2\ C} \tag{19}$$

where,

E = signal system efficiency, percent.

Efficiency increases when a larger portion of the cycle is being used for progression. The criteria provided in the PASSER II documentation for the interpretation of efficiency are reproduced in Table 5-1 (*11*).

Table 5-1. Evaluation Criteria for Efficiency.

Efficiency, percent	Progression Quality
0 to 12	Poor
13 to 24	Fair
25 to 36	Good
37 to 100	Great

The efficiency computed using Equation 19 is described as system efficiency because it considers the total bandwidth for both coordinated phases. The efficiency for just coordinated phase 2 (or phase 6) can be computed by dividing the phase 2 (or 6) bandwidth by the cycle length and multiplying by 100.

Attainability describes the portion of the critical green intervals that is available for progression. It is calculated as follows:

$$A = 100 \frac{B}{G_2 + G_6} \tag{20}$$

where,

A = signal system attainability, percent;
G_2 = critical phase 2 green interval duration, s; and
G_6 = critical phase 6 green interval duration, s.

Attainability describes whether the coordinated phases' green intervals are used effectively for progression. A low attainability suggests that a large interference is occurring at one or more intersections and that adjusting the offsets or left-turn phase sequence may be beneficial. An attainability of 100 percent suggests that bandwidth is limited only by the length of the critical green intervals and that all of the non-critical intersections are timed and located such that they do not interfere with the progressed movement. The criteria provided in the PASSER II documentation for the interpretation of attainability are reproduced in Table 5-2 (*11*).

Table 5-2. Evaluation Criteria for Attainability.

Attainability, percent	Evaluation
0 to 69	Major changes needed
70 to 99	Fine-tuning needed
99 to 100	Increase minimum green interval duration

The attainability computed using Equation 20 is described as system attainability because it considers the bandwidth for both coordinated phases. The attainability for coordinated phase 2 (or phase 6) can also be computed by dividing the phase 2 (or phase 6) bandwidth by the critical phase 2 (or phase 6) green interval length and multiplying by 100.

The system efficiency and attainability measures provided by TSCO are weighted based on a user-specified weight factor. In particular, the efficiency for each coordinated phase is used to compute a weighted average system efficiency, where the weight for phase 2 is specified by the analyst. A similar approach is used to compute a weighted average system attainability. If the specified weight is equal for both phases, then the computed efficiency and attainability are equal to the values obtained from Equations 19 and 20. If there is a preference for more of the bandwidth to be assigned to one phase, then the weight factor should be selected accordingly. In this manner, the timing plan with the largest efficiency and attainability will favor the preferred phase.

VALIDATION

This part of the chapter describes activities undertaken to evaluate the accuracy of the TSCO implementation of the bandwidth technique and its associated optimization algorithm. One objective of these activities was to verify that the worksheet calculations could correctly compute the bandwidth for typical streets and timing plans. The second objective was to verify that the TSCO optimization algorithm could compute the optimal timing plan.

Method

The validation process was based on a separate evaluation of three arterial street test beds. The characteristics of each test bed were based on those of an actual city street. They are generally described as:

- Intersection spacing is uneven and does not correspond with optimal spacing.
- Progression speed changes along the street.
- Left-turn phase sequence varies among the intersections in a single test bed, including lead-lead, lag-lag, lead-lag, lag-lead, and split phasing.

Five system cycle lengths were simulated for each of the three test beds to yield 15 different evaluation scenarios. Descriptions of the three test beds and scenarios are provided in Table 5-3.

Table 5-3. Validation Test Bed Characteristics.

Test Bed	Number of Signals	Speed, mph	Cycle Length (C), s [1]		Critical G/C, %		Left-Turn Phase Sequences [2]
			Low	High	Phase 2	Phase 6	
1	5	40	60	80	30	33	Lead-lead, lag-lead, lead-lag, split
2	6	35	80	160	43	43	Lead-lead, lag-lag, lead-lag, lag-lead
		45	80	160	43	43	
3	6	45	80	100	30	30	Lead-lead, lag-lead, lead-lag

Notes:
1 - Five cycle lengths were used for each street, in even increments between the low and high cycle lengths.
2 - One phase sequence was chosen for each intersection. This column provides a list of all sequences used on the street.

The evaluation involved three steps. First, each scenario was coded into the PASSER II simulation model. PASSER II was chosen for this evaluation because it optimizes signal timing using the same bandwidth technique as used in TSCO. The goal was to compare timing plans that were developed using PASSER II with those developed using TSCO (after removing any sensitivity to the influence of traffic volume). Thus, a nominal volume of ten vehicles per hour was coded for all intersection movements in PASSER II. The optimization algorithm of PASSER II was then exercised to obtain optimal coordination timing plans for the fifteen scenarios. The offsets and efficiencies for each timing plan were recorded.

During the second step, each scenario was coded into TSCO using the offsets generated by PASSER II. The resulting TSCO-calculated efficiencies were then compared to those obtained directly from PASSER II. The purpose of this step was to determine whether TSCO could produce the same measures of effectiveness as PASSER II.

When PASSER II calculates bandwidth, it allows the entire phase split (green, yellow, and red clearance) for phases 2 and 6 to be used, instead of just the green interval. To obtain a valid comparison between the two software tools, the PASSER II bandwidths were used to calculate efficiency as follows:

$$E = 100 \, \frac{B_2^{\prime} + B_6^{\prime} - 2Y - 2R_c}{2\ C} \tag{21}$$

where,

B_2' = phase 2 bandwidth from PASSER II, s;
B_6' = phase 6 bandwidth from PASSER II, s;
Y = yellow change interval for phases 2 and 6, s; and
R_c = red clearance interval for phases 2 and 6, s.

To facilitate this calculation, the same yellow and red clearance values were used for all signal phases. PASSER II reports bandwidths B_2' and B_6' rounded down to the nearest second. Thus, the PASSER II bandwidths were adjusted upward by 0.5 s to remove rounding bias before they were used in Equation 21.

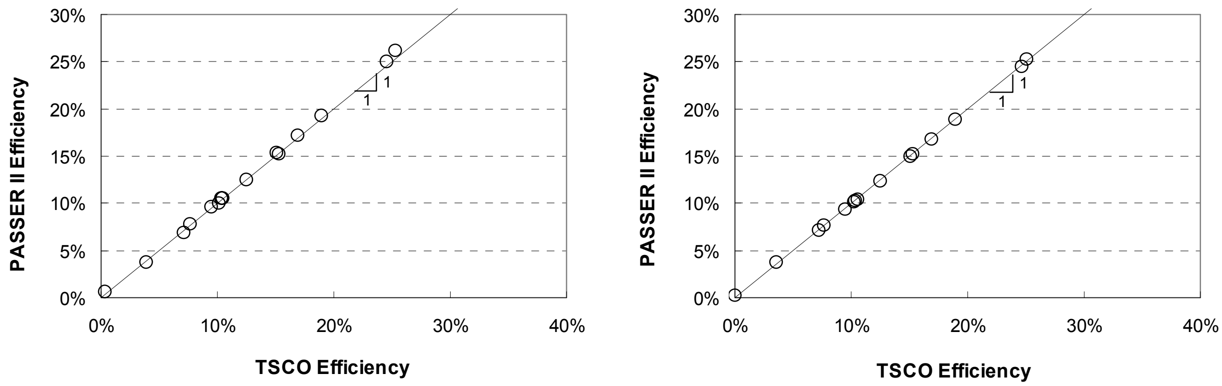
During the third step, the TSCO optimization algorithm was exercised to obtain a new set of optimal offsets for each scenario. The efficiencies associated with the TSCO optimized timing plans were compared to those obtained in step two (which were based on PASSER II offsets). The purpose of this comparison was to determine whether TSCO could identify the same optimal timing plans as PASSER II when given the same input conditions.

Results

Figure 5-3a shows the findings from the second evaluation step. Specifically, it compares the efficiencies obtained from PASSER II (in the first evaluation step) with those calculated from TSCO using the PASSER II offsets. Figure 5-3b shows the findings from the third evaluation step. It compares the efficiency based on the PASSER II solution with that obtained from TSCO. The lines shown in these figures represent the "$y = x$" line. Data points falling on this line represent cases when the compared efficiencies were equal.

In Figure 5-3a, the compared efficiency values are shown to be within 1 percent of each other. This value indicates that PASSER II and TSCO compute the same value of efficiency when they are given the same signal system and timing plan. In Figure 5-3b, the results are within 0.1 percent of each other, indicating that PASSER II and TSCO are equally effective at obtaining optimal timing plans for the 15 scenarios.

Best-fit trend lines are not shown in Figure 5-3. However, linear regression statistics were calculated, and neither of the associated regression relationships were significantly different from the "$y = x$" line. The subtle differences shown in Figure 5-3 indicate that PASSER II can identify slightly more efficient timing plans than TSCO under some conditions. This trend is likely a consequence of PASSER II's ability to generate offsets with a 0.1-s precision, while TSCO generates offsets to a 1-s precision.



a. Worksheet Calculation Evaluation. **b. Optimization Algorithm Evaluation.**

Figure 5-3. TSCO Algorithm Validation Data.

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APPENDIX A

USER'S MANUAL - TEXAS SIGNAL COORDINATION OPTIMIZER

APPENDIX A. USER'S MANUAL - TEXAS SIGNAL COORDINATION OPTIMIZER

OVERVIEW

This manual describes a software tool that can assist the practitioner in the development of a coordinated signal timing plan. This Microsoft Excel®-based software tool is called the Texas Signal Coordination Optimizer (TSCO). It can be used to evaluate and optimize the following basic settings that comprise a coordinated timing plan:

- cycle length,
- phase splits,
- offsets from system reference time, and
- left-turn phase sequence.

TSCO does not require extensive traffic data such as volumes and saturation flow rates, so the tool is most appropriate for cases when current traffic data are not available. When these data are available, more powerful software tools (e.g., PASSER II, TRANSYT-7F, and Synchro) should be considered because they are likely to produce more effective timing plans.

This manual is comprised of six parts. The first part provides an overview of the TSCO software. The second through sixth parts provide guidelines for using each of the five worksheets in TSCO.

SOFTWARE OVERVIEW

Worksheet Description

The TSCO software contains the following six worksheets:

- Welcome,
- Analysis,
- Splits,
- Volumes,
- Left-Turn Mode, and
- Preemption.

The Welcome worksheet is shown in Figure A-1. It contains an overview of the software's appearance, brief instructions, and citations to documentation to which the analyst should refer for a detailed description of the software's calculation methods and output. The tabs at the bottom of the Welcome worksheet are used to select each of the five remaining worksheets.

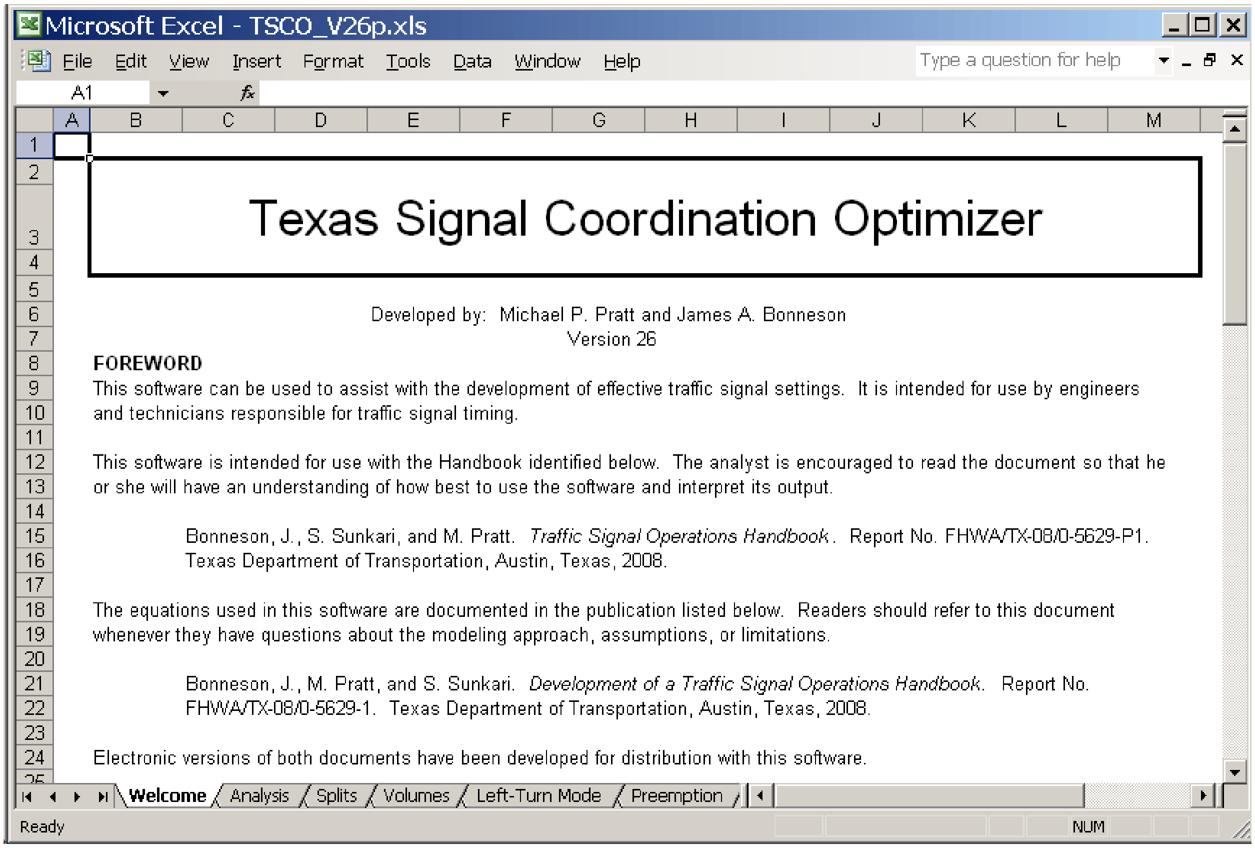


Figure A-1. TSCO Welcome Worksheet.

Worksheet Appearance and Control

A color coding scheme is used in the worksheets to indicate the purpose of each cell and to direct the analyst toward cells that need to be manipulated.

Blue cells contain input data that describe the intersection being analyzed. These cells will be manipulated most often for typical worksheet applications. Additionally, some types of input data or preferences are entered using drop-down menus, check boxes, or spin buttons instead of blue cells. The analyst may manipulate these controls as appropriate to describe the analysis conditions.

Yellow cells contain calibration factors that can be adjusted based on local conditions. Changes to these cells should be based on field data collected within the agency's jurisdiction.

Green cells contain default values for certain parameters. These values should be reasonable for most applications, but may be changed to reflect local conditions or practices, based on local field data or agency policy.

Purple cells contain key output data. White cells contain intermediate calculations or guidance (e.g., advisory messages) for the analyst.

Only values in blue, yellow, or green cells can be changed. A couple of the worksheets also contain a button or two in the upper left corner of the worksheet. This button must be clicked to invoke the calculation routines when input data are changed.

ANALYSIS WORKSHEET

This part of the appendix provides a detailed description of the Analysis worksheet. This worksheet can be used to evaluate or optimize a coordinated signal timing plan for a signalized arterial street. The description in this part identifies the input data requirements, output data presentation and interpretation, procedures for optimization, and various controls for refining the analysis.

Input Data Requirements

The Analysis worksheet models the street as a series of nodes separated by segments. Up to ten nodes (separated by nine street segments) can be modeled. A node can represent the location of a signalized intersection, a change in traffic speed, or both. The required input data are listed in Table A-1. If the node represents a mid-block speed change (i.e., no signal is present), only the data that are underlined in this table need to be entered.

Table A-1. Analysis Worksheet Input Data Requirements.

Data Category	Item [1]	Scope
Node and segment description	<u>Distance coordinate, ft</u>	Each node
	<u>Signal presence</u>	Each node
	<u>Progression speed, mph</u>	Each segment
Signal timing	<u>System cycle length, s</u>	Entire street
	Phase split, percent of system cycle	Each signal (major-street phases)
	Change period (yellow + red clearance), s	Each signal (major-street phases)
	Left-turn phase sequence	Each signal (major-street phases)
	Offset, s	Each signal (major-street phases)

Note:
1 - Data required to describe a mid-block speed change are indicated by underline.

Node and Segment Description Data

The labeling scheme for nodes and segments is shown in Figure A-2. The nodes are numbered 1 through 10 and the segments are labeled A through I. A description (e.g., cross street name) of each node may be entered if desired. Check boxes are provided to indicate the presence of a signal at a specific node. A distance coordinate *x* for each node must be entered. It represents the distance between the subject intersection and the first intersection, as represented by the intersection with the lowest node number. TSCO calculates segment length from the distance coordinates. If fewer than ten nodes are needed, the distance coordinate cells for unused nodes should be left blank.

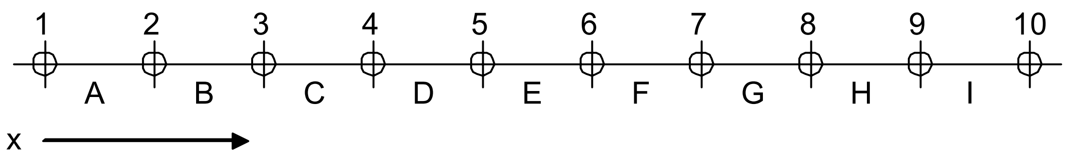


Figure A-2. Node and Segment Labeling Scheme.

The "node description" data entry cells are shown in Figure A-3. Example data are shown in the figure. The entered data describe Main Street, an example street with five signalized intersections. The third signal (Spring Road) is located 2260 ft from the first signal (5th Avenue).

Signal Timing Data											
	Data Description	Node Data									
Search	Node Description:	5th Ave	College Rd	Spring Rd	Forest Dr	Del Mar St					
	Signal present? (Check = yes):	1 ☑	2 ☑	3 ☑	4 ☑	5 ☑	0 ☐	0 ☐	0 ☐	0 ☐	0 ☐
Tweak	Distance coordinate (x), ft:	0	1300	2260	3330	3950					
	Offset, s:	0	34	52	5	7					

Figure A-3. Node Description Data Entry Cells.

The analyst must provide a progression speed for each segment. This speed represents the average running speed of platooned vehicles as they progress through the coordinated signals. It should be representative of *both* travel directions along the segment. Note that this speed may differ from the posted speed limit. The progression speed for a segment is entered in the same column as the upstream node that bounds this segment. For example, the progression speed for segment A is entered in the same column as is the data for node 1.

Signal Timing Data

The analyst must provide a system cycle length. This value is input in the upper-right corner of the Analysis worksheet.

The phase-specific signal timing data entry cells are shown in Figure A-4. For each signalized intersection, the analyst must enter phase splits (i.e., the sum of the green, yellow, and red clearance intervals) and change period (i.e., the sum of the yellow and red clearance intervals) for the major-street phases. The left-turn movement phases are numbered 1 and 5. The through movement phases are numbered 2 and 6. To facilitate the evaluation of different cycle lengths, phase splits are entered as percentage of cycle length.

TSCO displays the green interval duration that is calculated from the entered phase split and change period data. The analyst can review these interval durations for accuracy. For example, the phase 2 green interval at the second signal (i.e., College Road) in Figure A-4 is shown to be 32 s.

Signal Timing Data											
	Data Description	Node Data									
Search	Node Description:	5th Ave	College Rd	Spring Rd	Forest Dr	Del Mar St					
	Signal present? (Check = yes):	1 ☑	2 ☑	3 ☑	4 ☑	5 ☑	0 ☐	0 ☐	0 ☐	0 ☐	0 ☐
Tweak	Distance coordinate (x), ft:	0	1300	2260	3330	3950					
	Offset, s:	0	34	52	5	7					
Phase 1	Phase split, % of cycle:	12%	20%	33%	10%	18%					
Westbound	Green interval, s:	4	9	17	3	8	0	0	0	0	0
	Change period (Y + RC), s:	4	4	4	4	4					
Left Turn	Phase sequence:	Lead	Lag	Lead	Lag	Lead	Lead	Lag	Lead	Lag	Lag
Phase 2	Phase split, % of cycle:	52%	56%	30%	34%	44%					
Eastbound	Green interval, s:	30	32	16	18	25	0	0	0	0	0
	Change period (Y + RC), s:	4	4	4	4	4					
Phase 5	Phase split, % of cycle:	20%	20%	30%	10%	12%					
Eastbound	Green interval, s:	9	9	16	3	4	0	0	0	0	0
	Change period (Y + RC), s:	4	4	4	4	4					
Left Turn	Phase sequence:	Lead	Lead	Lag	Lead	Lag	Lag	Lead	Lag	Lead	Lag
Phase 6	Phase split, % of cycle:	44%	56%	33%	34%	50%					
Westbound	Green interval, s:	25	32	17	18	29	0	0	0	0	0
	Change period (Y + RC), s:	4	4	4	4	4					

Figure A-4. Phase-Specific Signal Timing Data.

The analyst must also indicate the phase sequence for the major-street movements. Using the drop-down menus provided, the analyst can indicate whether the left-turn phase leads or lags the opposing through phase.

To code split phasing, the analyst must choose “lead” for the left-turn phase that occurs first and “lag” for the left-turn phase that occurs second. The phase split and change period values must be equal for phases 2 and 5, and also for phases 1 and 6. In the example data shown in Figure A-4, the third signal (i.e., Spring Road) has been coded with split phasing, with phases 1 and 6 leading.

If a protected left-turn phase is omitted, enter blank values for the omitted phase split and the change period (i.e., delete any numerical values in the blue cells for the omitted phase).

The analyst may also enter an offset for each signalized intersection. The reference time for offset is defined as the start of the phase 2 green interval. It is recommended that the start of phase 2 green at intersection 1 be used as the system reference time (i.e., use an offset of 0 s for intersection 1); however, TSCO does not require adherence to this recommendation. In the example data shown in Figure A-4, the fifth signal (i.e., Del Mar Street) has an offset of 7 s, indicating that the phase 2 green interval starts 7 s later at the fifth signal than at the first signal.

If signal timing data are entered for a node but the signal presence check box is left unchecked, the signal timing data for the node will not be used in any of the worksheet calculations. Using this feature, the effect of adding a signal to a coordinated signal system can be evaluated. Initially, the signal timing data and node coordinate for the location of the proposed new signal are input along with similar data for the existing intersections. The signal presence box is left unchecked for the new signal. The signal system is evaluated and the performance measures noted. Then, the box is checked and the system is re-evaluated. A comparison of the performance measures from the two evaluations indicates the impact of the proposed signal.

Output Data

Time-Space Diagram

TSCO produces a time-space diagram for the subject street. The time-space diagram is arranged with distance on the *x*-axis and time on the *y*-axis. An example time-space diagram from TSCO is shown in Figure A-5. Two multicolored vertical lines are used to portray the signalization at each intersection. These lines are referred to hereafter as "cycle bars." For each pair of cycle bars, the bar on the left corresponds to phases 2 and 5. The bar on the right corresponds to phases 1 and 6.

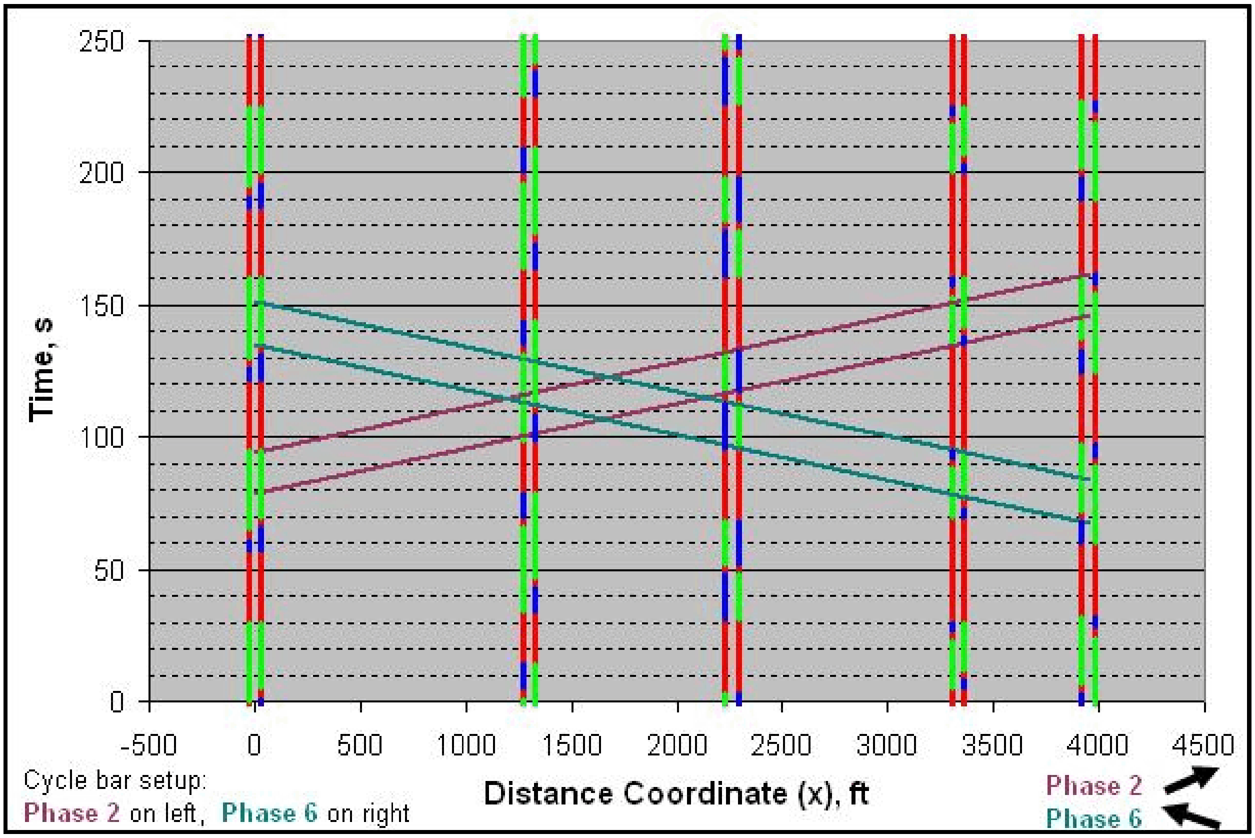


Figure A-5. Time-Space Diagram.

A more detailed examination of cycle bars at one intersection is provided in Figure A-6. The cycle bars for each intersection use green to represent the phase 2 and 6 green intervals, blue to represent the phase 1 and 5 green intervals, and red to represent all other time in the cycle. Within each cycle bar, ring 1 (including phases 1 and 2) is shown on the left, and ring 2 (including phases 5 and 6) is shown on the right.

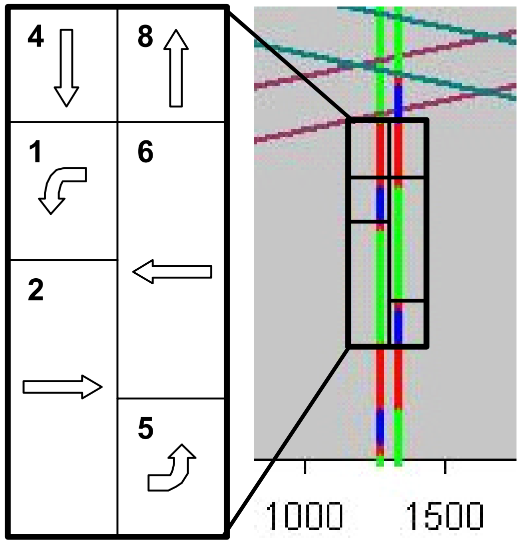


Figure A-6. Time-Space Diagram Cycle Bar Details.

Figure A-5 also shows the progression bands for both travel directions along the street. The phase 2 band is colored purple. It slopes upward from left to right. The phase 6 band is colored blue-green. It slopes downward from left to right. The vertical distance between the two parallel lines that form a progression band is defined as the bandwidth. Bandwidth is measured in units of time (e.g., seconds).

By inspecting the time-space diagram, the analyst can verify that the signal timing settings have been entered correctly and determine at a glance whether good progression would be accommodated by the timing plan. The analyst may also choose to try different left-turn phase sequences based on the location of the green intervals with respect to the progression bands.

Controls are provided so the analyst can adjust the appearance of the time-space diagram. These controls are shown in Figure A-7.

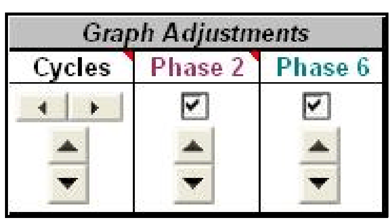


Figure A-7. Graph Adjustment Controls.

The horizontal spacing between the cycle bars can be changed. Having a small amount of space between the bars is helpful, but the amount of space needed changes proportionally if the scale of the *x*-axis is changed. This adjustment is made using the horizontal spin button in the "Cycles" box.

The vertical position of the green intervals can be adjusted. This adjustment may be needed if an exceptionally small cycle length is used and green intervals cannot be seen at the top or bottom of the diagram. This adjustment is made using the vertical spin button in the "Cycles" box.

The position of each progression band can be adjusted vertically. Depending on the chosen offsets and cycle length, the progression bands may occasionally plot in cycles above or below the displayed 250-s range, or portions of the bands near the left and right sides of the diagram may be obscured from view. These adjustments are made using the spin buttons in the "Phase 2" and "Phase 6" boxes.

The cycle bars and progression bands for phases 2 and 6 may be deactivated if desired. Depending on the visual complexity of the signal system, it may be helpful to view the phase 2 and 6 bands separately. These adjustments are made using the check boxes in the "Phase 2" and "Phase 6" boxes.

The *y*-axis and *x*-axis parameters (i.e., minimum, maximum, and scale) of the time-space diagram are automatically adjusted for best viewing. This adjustment is made each time the node coordinates are changed.

By default, the worksheet is set to give equal weight to phases 2 and 6 when evaluating coordination timing plans. If desired, the analyst may specify more or less weight for phase 2, relative to phase 6. The desired weight relationship can be entered in the "Phase 2 Weight" drop-down box just below the Graph Adjustment Controls in the lower right corner of the worksheet.

The "Phase 2 Weight" setting is useful for cases when traffic volumes are higher in one direction than the other. When an unequal weight is used for phases 2 and 6, the calculated system efficiency and attainability are based on a weighted average of the bandwidth for the two coordinated phases.

Measures of Effectiveness

TSCO provides measures of effectiveness for the coordinated signal system as well as for the two coordinated phases separately. These measures include total bandwidth, efficiency, and attainability. Equations for their calculation are described in Chapter 5.

The concept of bandwidth, as defined by a progression band, was previously described in reference to Figure A-5. Each travel direction along a street has a bandwidth. The sum of these two bandwidths equals the total bandwidth for the signal system. In general, a wider progression band denotes a better timing plan relative to the quality of traffic flow along the street. Bandwidth usually increases with cycle length.

Efficiency is another measure of system performance. It represents the portion of the system cycle length that is used for progression. Efficiency usually increases with cycle length; however, it may also decrease if a smaller portion of the cycle is being used for progression. The criteria provided in the PASSER II documentation for the interpretation of efficiency are reproduced in Table A-2 (*1*).

Table A-2. Evaluation Criteria for Efficiency.

Efficiency, percent	Progression Quality
0 to 12	Poor
13 to 24	Fair
25 to 36	Good
37 to 100	Great

Efficiency is typically evaluated for the signal system using the total bandwidth. However, it can also be computed for each coordinated phase. In this situation, Table A-2 would be used to separately evaluate each travel direction.

Attainability describes the portion of the critical green interval that is available for progression. It describes whether the green interval is used effectively for progression. The criteria provided in the PASSER II for the interpretation of attainability are reproduced in Table A-3 (*1*).

Table A-3. Evaluation Criteria for Attainability.

Attainability, percent	Evaluation
0 to 69	Major changes needed
70 to 99	Fine-tuning needed
99 to 100	Increase minimum green interval duration

A low attainability (say, less than 70 percent) indicates that a large interference is occurring at one or more intersections, and that adjusting the offsets or left-turn phase sequence may be beneficial. An attainability of 100 percent suggests that bandwidth is limited only by the length of the critical green intervals, and that all of the non-critical intersections are timed and located such that they do not interfere with the progressed movement.

The aforementioned measures of effectiveness are reported in the lower right corner of the Analysis worksheet. Performance measures for an example street system are shown in Figure A-8. System attainability is shown to be 98.6 percent which indicates that some fine-tuning may be needed. System efficiency is shown to be 25 percent which indicates that the progression is of good quality. These system measures reflect any directional weighting that may have been specified by the analyst. The phase measures of effectiveness shown in the bottom of Figure A-8 do not include the effect of any weighting factors.

System Measures of Effectiveness		
Bandwidth, s:	32.5	
Weighted Efficiency:	25.0%	Good
Weighted Attainability:	98.6%	Fine tune

Phase Measures of Effectiveness		
	Phase 2	Phase 6
Critical intersection:	3	3
Interference, s:	0.0	0.5
Bandwidth, s:	16.0	16.5
Efficiency:	24.6%	25.4%
Attainability:	100.0%	97.2%

Figure A-8. Example System Measures of Effectiveness.

Advisory Messages and Checks

The Advisory Message and Check section of the worksheet is shown in Figure A-9. Advisory messages are generated when the analyst omits phase split data for phases 2 or 6 or progression speed for a segment. They are also generated when the phase splits for phases 1 and 2 do not equal those for phases 5 and 6. In Figure A-9, intersection 1 (i.e., 5th Avenue) is noted as missing numeric data, and intersection 4 (i.e., Forest Drive) is noted as having a barrier conflict. A list of the advisory messages and possible solutions is provided in Table A-4.

	Data Description	Node Data									
Search	Node Description:	5th Ave	College Rd	Spring Rd	Forest Dr	Del Mar St					
	Signal present? (Check = yes):	1 ☑	2 ☑	3 ☑	4 ☑	5 ☑	0 ☐	0 ☐	0 ☐	0 ☐	0 ☐
Tweak	Distance coordinate (x), ft:	0	1300	2260	3330	3950					
	Offset, s:	0	34	52	5	7					
Advisory Messages and Checks											
%Complete 100	Cross street phase splits, s:	23	16	24	36	24					
No. Iterations	Numeric data missing above.	See Note									
64	Barrier conflict check: Splits 1+2 should equal splits 5+6.				See Note						

Figure A-9. Advisory Messages and Checks.

Table A-4. Advisory Messages and Possible Solutions.

Advisory Messages	Possible Solutions
Numeric data missing above.	• Enter a phase split for phase 2 or phase 6. • Enter a change period for phase 2 or phase 6. • Enter a progression speed for the segment.
Splits 1+ 2 should equal splits 5+6.	Adjust the phase splits for phases 1, 2, 5, and 6. The sum of the phase splits for phases 1 and 2 must equal that for phases 5 and 6.

Automatic Optimization Settings

The analyst can specify the offset precision, which is the increment by which the offsets are adjusted when the optimization algorithm is exercised. Acceptable values include 1, 2, 3, 4, or 5 s. A value of 1 s is recommended. A higher value will yield a less precise solution but reduce computation time by limiting the number of offset values tested. Similarly, the analyst can specify the cycle length precision, which is the increment by which the system cycle length is adjusted when the optimization algorithm is exercised. Any integer value between 1 and 20 s is acceptable. A value of 5 s is recommended. The analyst can also indicate (by un-checking the appropriate boxes) if one or more of the intersections' offsets should not be changed. These settings are entered near the bottom of the Analysis worksheet.

The system cycle length is entered in the upper right corner of the Analysis worksheet. To specify the range of desired cycle lengths for optimization, enter the minimum desired cycle length in the cell for "system cycle length." Just below this cell, enter the maximum cycle length. If cycle length optimization is not desired, then enter the same value in both cells.

Optimization Procedures

TSCO provides calculations to assist with the development of an effective timing plan. The analyst can manually adjust the offsets to obtain a desired timing plan, perhaps by starting with a timing plan that has already been implemented in the field. Alternatively, the analyst may let TSCO optimize offsets and cycle lengths automatically.

Manual Adjustment Method

If the goal is to evaluate or improve an existing timing plan, the analyst can choose to adjust the timing plan manually to determine whether a greater bandwidth could be attained. To assist with manual offset adjustment, TSCO provides guidance indicating which intersections tend to cause the most bandwidth loss for each coordinated phase. This guidance is in the form of an indication to increase or decrease the offset at a particular intersection to improve the bandwidth. This guidance is shown in the bottom portion of Figure A-10. Such adjustments could also be made based on interpretation of the time-space diagram; however, visual inspection of the diagram may not be sufficient to determine which intersections cause the most bandwidth loss.

	Data Description	Node Data									
Search	Node Description:	5th Ave	College Rd	Spring Rd	Forest Dr	Del Mar St					
	Signal present? (Check = yes):	1 ☑	2 ☑	3 ☑	4 ☑	5 ☑	0 ☐	0 ☐	0 ☐	0 ☐	0 ☐
Tweak	Distance coordinate (x), ft:	0	1300	2260	3330	3950					
	Offset, s:	0	34	52	5	7					
Manual Offset Adjustment Guidance											
	To increase phase 2 bandwidth										
	To increase phase 6 bandwidth	Decrease		Increase							

Figure A-10. Manual Offset Adjustment Guidance.

For the example street, 100 percent attainability has been achieved for phase 2, so no guidance is provided in Figure A-10 for offset adjustments to increase phase 2 bandwidth. A small amount of interference (0.5 s) occurs for phase 6. It could be reduced by decreasing the first signal's offset or increasing the third signal's offset.

Automatic Optimization Method

Three types of signal timing optimization can be performed with TSCO, they are:

1. Starting with a candidate timing plan (i.e., phase splits, left-turn phase sequences, cycle length, and offsets), determine whether minor offset adjustments increase the bandwidth.
2. Starting with phase splits, left-turn phase sequences, and cycle length, identify the optimal offsets.
3. Starting with phase splits and left-turn phase sequences, identify the optimal offsets and cycle length.

For a Type 1 optimization, the analyst will likely start with an existing timing plan from the field and try to improve its effectiveness, perhaps in response to an anticipated change to the street or its signals. Such changes might include adding a signal, implementing a left-turn phase, or changing the left-turn phase sequence at one or more signals. To conduct a Type 1 optimization, enter the input data, including offsets, and click the "Tweak" button. The optimization algorithm will search for a better timing plan using the input plan as a starting point.

For a Type 2 optimization, the analyst may desire to search for improvements to an existing timing plan. Alternatively, the analyst may want to develop a new timing plan for an existing street. A Type 2 optimization is more thorough than a Type 1 optimization because it generates and tests a larger set of candidate timing plans. To conduct a Type 2 optimization, enter all input data and click the "Search" button. The analyst may begin with a set of candidate offsets or leave the offset entry cells blank. The optimization algorithm will identify and evaluate a large number of feasible offset values.

For a Type 3 optimization, the analyst may desire to evaluate different cycle lengths for the street. Alternatively, the analyst may want to develop a new timing plan for an existing street for which a system cycle length has not been chosen. To conduct a Type 3 optimization, enter all input data and click the "Search" button. The analyst may begin with a set of candidate offsets or leave the offset entry cells blank, but must specify a cycle length range. The optimization algorithm will identify and evaluate a large number of feasible offset values. This process is repeated for all cycle lengths within the specified range.

For all types of optimization, the algorithm will update the worksheet to show the timing plan that yields the greatest efficiency while providing bandwidth for both coordinated phases. If no such timing plan is identified, it will leave the worksheet unchanged.

While the optimization algorithm runs, it periodically updates the screen and provides the analyst with status information to indicate its progress. This information is shown on the left side of Figure A-9. It is also repeated just to the left of the Graph Adjustment Controls. During a Type 3 optimization, the "% Complete" indicator resets to "0" at the start of each new cycle length that is being evaluated.

SPLITS WORKSHEET

The Splits worksheet is used to calculate effective phase splits for coordinated signal operation. The methodology was developed by Bonneson and Fontaine (*2*). This part of the appendix provides a brief description of the worksheet. The description includes input data requirements and output data presentation and interpretation. More discussion about this worksheet is provided in the *Traffic Signal Operations Handbook* (*3*).

Input Data Requirements

The Splits worksheet requires the following input data:

- cycle length,
- left-turn operational mode (i.e., permissive or protected left-turn phase),
- change period (i.e., yellow change plus red clearance intervals),
- minimum green setting,
- turn movement volume, and
- approach lane allocations to left-turn and through movements.

The input cells for this worksheet are shown in Figure A-11. Drop-down menus are provided so the analyst can specify the left-turn operational mode and phasing. All other data are entered as values in the appropriate cell.

Phase Split Calculation Worksheet								
General Information								
Location:	***Main St. & Peachtree Drive***				Analysis Period:		***7:00 to 9:00 am***	
Cycle Length (C), s: **100**	Eastbound & Westbound Phasing:				Northbound & Southbound Phasing:			
Phase 2: EB	LT Phase & TH Phase				LT & TH in same phase			
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**
Change Period and Minimum Green								
Yellow + red clearance (Y_i), s	**5**	**5**	**5**	**5**		**5**		**5**
Minimum green ($G_{m,i}$), s	**8**	**10**	**8**	**10**		**16**		**16**

Figure A-11. Splits Worksheet Data Entry Cells.

Output Data

Once all of the data have been entered, the computed phase splits are automatically calculated and shown in the bottom portion of the worksheet. Figure A-12 shows the output section of the worksheet based on the data shown in Figure A-11. The computed phase splits are provided in units of "seconds" and "percent of cycle." The latter units are more useful when using the Analysis worksheet.

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
Computed Phase Splits								
Phase split (T_i), s {see note 6}	**13**	**61**	**18**	**66**	**0**	**21**	**0**	**21**
Phase split, % [= 100 T_i/C]	**13**	**61**	**18**	**66**	**0**	**21**	**0**	**21**
Checks:								

Figure A-12. Splits Worksheet Output Data.

VOLUMES WORKSHEET

The Volumes worksheet calculates approximate turn movement volumes for an intersection. It is useful when actual turn movement counts are not available. The methodology was developed by Van Zuylen (*4*). This part of the appendix provides a description of the worksheet. The description includes input data requirements, output data presentation, and output data interpretation.

Input Data Requirements

The Volumes worksheet calculates turn movement volumes based on user-provided traffic and roadway characteristics. The required input data are listed in Table A-5. Also listed in this table are the recommended calibration factors. Analysts may wish to adjust these factors based on local traffic patterns to improve the accuracy of the estimated volumes.

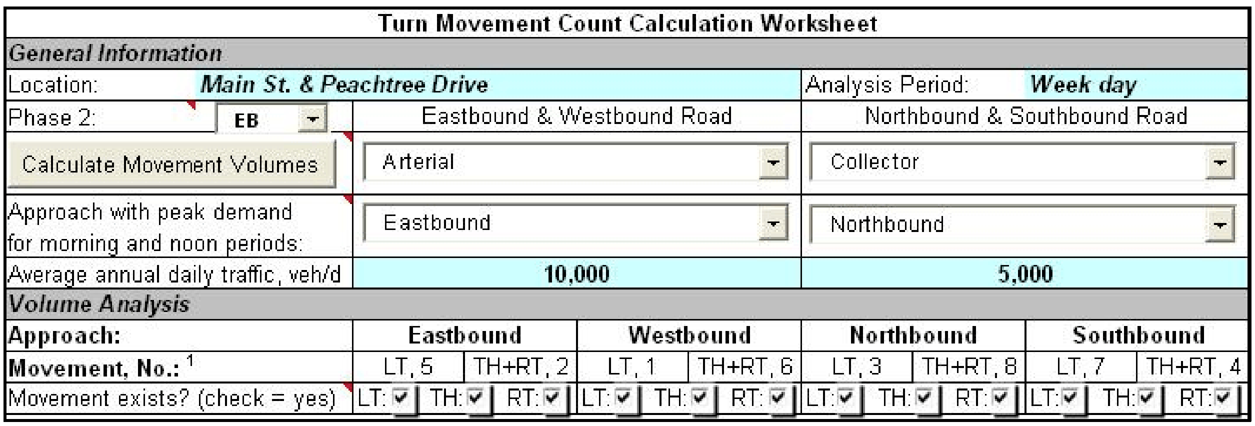
Table A-5. Volumes Worksheet Input Data Requirements.

Data Category	Item	Scope
Roadway description	Functional classification	Both intersecting roads
	Intersection configuration	All movements
Traffic description	Approach with peak demand	Both intersecting roads
	Average annual daily traffic (AADT), veh/d	Both intersecting roads
Calibration factors	Traffic distribution by time period	By time of day
	Peak period directional distribution	By time of day
	Left-turn percentage	By functional classification
	Right-turn percentage	By functional classification

The calculations in the Volumes worksheet are implemented in computer code that is activated each time the "Calculate Movement Volumes" button is clicked. Thus, if any changes are made to the input data, the analyst must click this button to ensure that the changes are reflected in the estimated turn movement volumes. This button is located near the upper-left corner of the worksheet.

Roadway Description Data

The input cells for the roadway and traffic description data are shown in Figure A-13. Drop-down menus are provided so the analyst can specify the functional classification for each intersecting road. Drop-down menus are also provided to specify the approach direction that experiences peak demand. The analyst must provide the AADT for each roadway.



Turn Movement Count Calculation Worksheet								
General Information								
Location:	*Main St. & Peachtree Drive*				Analysis Period:		*Week day*	
Phase 2: EB	Eastbound & Westbound Road				Northbound & Southbound Road			
Calculate Movement Volumes	Arterial				Collector			
Approach with peak demand for morning and noon periods:	Eastbound				Northbound			
Average annual daily traffic, veh/d	10,000				5,000			
Volume Analysis								
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
Movement exists? (check = yes)	LT: ☑ TH: ☑	RT: ☑	LT: ☑ TH: ☑	RT: ☑	LT: ☑ TH: ☑	RT: ☑	LT: ☑ TH: ☑	RT: ☑

Figure A-13. Volumes Worksheet Data Entry Cells.

Check boxes are provided so the analyst can indicate which movements (i.e., left turn, through, and right turn) are accommodated on each approach. These check boxes are shown in the bottom row of Figure A-13. All check boxes are checked by default. This condition corresponds to a conventional four-leg intersection that accommodates all movements. A diagram is provided in the worksheet to illustrate the correct check box configuration for the specification of a "T" (i.e., three leg) intersection.

Calibration Factors

The input cells for the calibration factors are shown in Figure A-14. The default values for the calibration factors are based on previous research (*5, 6*) and will yield reasonable turn movement volumes for most intersections. The analyst may adjust these values to more accurately reflect local traffic patterns. Adjustments should be based on actual traffic counts taken at intersections in the agency's jurisdiction.

Calibration Factors						
Volume Distribution						
Period	Percent of AADT[2]	Percent Peak Dir.		Facility Type	Percent Left Turn[2]	Percent Right Turn[2]
Morning Peak Period	6.4	60		Central Business District	10	12
Mid-Morning Period	5.0	50		Arterial to Arterial	12	12
Noon Peak Period	5.8	50		Arterial to Collector	4	5
Mid-Afternoon Period	5.7	50		Collector to Arterial	30	32
Evening Peak Period	7.9	60		Collector to Collector	10	20

Figure A-14. Volumes Worksheet Calibration Factors.

Output Data

Volume Analysis

The Volumes worksheet provides an estimate of the volume for each movement at the intersection. One estimate is provided for each of the following five time periods:

- morning peak period,
- mid-morning period,
- noon peak period,
- mid-afternoon period, and
- evening peak period.

The volume analysis cells are shown in Figure A-15. Each purple row corresponds to one time period, and each column corresponds to one movement (i.e., a left-turn movement or a combined through-and-right-turn movement) at the intersection. Volume distribution factors and approach turn movement volumes are also provided.

Note the Advisory Messages box at the bottom of Figure A-15. The calculations in the Volumes worksheet are iterative in nature such that the calculation algorithm must be re-run every time a change is made to the input data. If any input data are changed, a message reminding the analyst to click the "Calculate Movement Volumes" button will appear in the Advisory Messages box. If the button is not clicked, the output data will not reflect the changes to the input data.

Turn Movement Volumes

The Volumes worksheet also provides the turn movement volumes (in veh/hr) for the left-turn, through, and right-turn volumes. The box containing these data is shown in Figure A-16.

Volume Analysis								
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
Movement exists? (check = yes)	LT: ☑	TH: ☑ RT: ☑	LT: ☑	TH: ☑ RT: ☑	LT: ☑	TH: ☑ RT: ☑	LT: ☑	TH: ☑ RT: ☑
Morning Peak Period								
Volume distribution factor:	60		40		60		40	
Approach volume, veh/h	384		256		192		128	
Volume (v_i), veh/h (i = 1,2,3,...8)	**36**	**348**	**21**	**235**	**30**	**162**	**29**	**99**
Mid-Morning Period								
Volume distribution factor:	50		50		50		50	
Approach volume, veh/h	250		250		125		125	
Volume (v_i), veh/h (i = 1,2,3,...8)	**22**	**228**	**22**	**228**	**24**	**101**	**24**	**101**
Noon Peak Period								
Volume distribution factor:	50		50		50		50	
Approach volume, veh/h	290		290		145		145	
Volume (v_i), veh/h (i = 1,2,3,...8)	**25**	**265**	**25**	**265**	**28**	**117**	**28**	**117**
Mid-Afternoon Period								
Volume distribution factor:	50		50		50		50	
Approach volume, veh/h	285		285		143		143	
Volume (v_i), veh/h (i = 1,2,3,...8)	**25**	**260**	**25**	**260**	**27**	**115**	**27**	**115**
Evening Peak Period								
Volume distribution factor:	40		60		40		60	
Approach volume, veh/h	316		474		158		237	
Volume (v_i), veh/h (i = 1,2,3,...8)	**26**	**290**	**44**	**430**	**36**	**122**	**38**	**199**
Advisory Messages								
.								

Figure A-15. Volumes Worksheet Output Data.

Turn Movement Volumes												
	Eastbound			Westbound			Northbound			Southbound		
Period	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Morning Peak Period	36	315	34	21	200	35	30	122	40	29	73	26
Mid-Morning Period	22	201	27	22	201	27	24	76	26	24	76	26
Noon Peak Period	25	233	32	25	233	32	28	88	30	28	88	30
Mid-Afternoon Period	25	229	31	25	229	31	27	86	29	27	86	29
Evening Peak Period	26	247	43	44	388	42	36	90	32	38	150	49

Figure A-16. Volumes Worksheet Turn Movement Volumes.

LEFT-TURN MODE WORKSHEET

The Left-Turn Mode worksheet is used to determine the most appropriate left-turn mode for each intersection approach. The methodology was developed by Bonneson and Fontaine (2). This part of the appendix provides a brief description of the worksheet. The description includes input data requirements and output data presentation. More discussion about the rules and criteria used in this worksheet is provided in the *Traffic Signal Operations Handbook* (3).

Input Data Requirements

The Left-Turn Mode worksheet requires the following input data:

- cycle length,
- turn movement volume,
- approach lane allocations to left-turn and through movements,
- crash history (i.e., left-turn related crash counts),
- approach speed, and
- adequacy of sight-distance for the left-turning driver.

The input cells for this worksheet are shown in Figure A-17. Spin buttons are provided so the analyst can specify the time period corresponding to the crash counts. Drop-down lists are also provided to indicate whether sight distance is adequate. All other data are entered as values in the appropriate cell.

Left-Turn Operational Mode Calculation Worksheet								
General Information								
Location:	***Main St. & Peachtree Drive***				Analysis Period:		***7:00 to 9:00 am***	
Cycle Length (C), s:	**100**							
Phase 2:	EB							
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, veh/h	**105**	**502**	**201**	**806**	**93**	**408**	**57**	**104**
Lanes	**1**	**2**	**1**	**2**	**0**	**2**	**0**	**1**
Crash History								
Left-turn related crashes	**4**		**3**		**2**		**1**	
Time period for crashes, years:	**1**				**1**			
Speed and Sight Distance								
Approach speed, mph		**45**		**45**		**45**		**45**
Minimum sight distance (SDc), ft:	360		360		360		360	
Is sight distance for the left-turn driver adequate?	Yes		Yes		Yes		Yes	

Figure A-17. Left-Turn Mode Worksheet Data Entry Cells.

Output Data

Once all of the data have been entered, the suggested left-turn model is determined. a suggested left-turn mode is automatically determined and shown in the middle of the worksheet. Figure A-18 shows the output section of the worksheet based on the data shown in Figure A-17.

Analysis Results				
Suggested left-turn mode:	**Protected-Permissive**	**Protected-Permissive**	**Permissive**	**Permissive**

Figure A-18. Left-Turn Mode Worksheet Output Data.

PREEMPTION WORKSHEET

The Preemption worksheet is used to calculate time requirements for traffic signal preemption at highway-rail grade crossings. The methodology was developed by Engelbrecht et al. (*7*). This part of the appendix provides a brief description of the worksheet. The description includes input data requirements and output data presentation. More discussion about this worksheet is provided in the *Traffic Signal Operations Handbook* (*3*). A detailed discussion and guidelines for using this worksheet are provided by Engelbrecht et al. (*7*).

Input Data Requirements

The Preemption worksheet requires the following input data:

- controller response time to preempt,
- yellow change interval of worst-case conflicting vehicle phase,
- red clearance interval of worst-case conflicting vehicle phase,
- pedestrian clearance time during right-of-way transfer,
- yellow change interval of worst-case pedestrian phase,
- red clearance interval of worst-case pedestrian phase,
- clear storage distance between stop line and railroad tracks,
- minimum track clearance distance,
- clearance time (from railroad),
- advance preemption time (if used), and
- distance from center of gate support post to nearest side of the design vehicle.

The input cells for this worksheet are shown in Figure A-19. Drop-down lists are provided to indicate the design vehicle, approach grade, and warning time variability. All other data are entered as values in the appropriate cell.

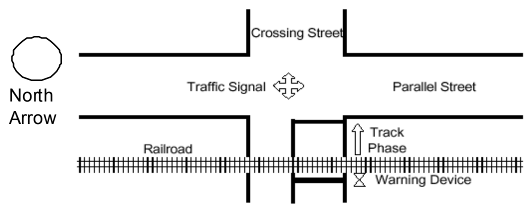
Output Data

Once all of the data have been entered, the preemption settings are automatically determined and shown on the right side of the worksheet. Figure A-19 also shows the output sections of the worksheet. The following values are calculated by the worksheet:

- right-of-way transfer time,
- queue clearance time,
- maximum preemption time,
- additional warning time required from railroad,
- track clearance green interval, and
- advance preemption time required to avoid design vehicle-gate interaction.

GUIDE FOR DETERMINING TIME REQUIREMENTS FOR
TRAFFIC SIGNAL PREEMPTION AT HIGHWAY-RAIL GRADE CROSSINGS [1]

General Information			
City:		Date:	11/10/2008
County:		Completed by:	
District:		District Approval:	

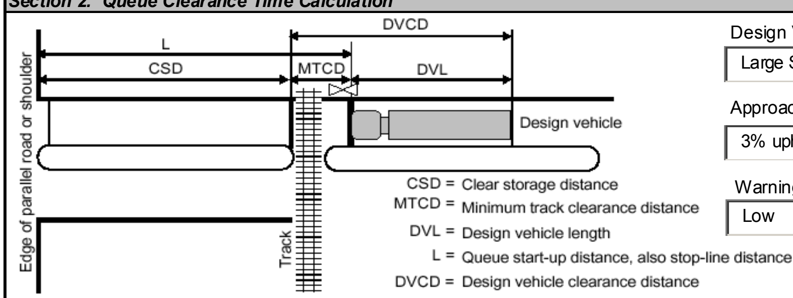


Parallel Street Name:	
Crossing Street Name:	

Note: Cells with green shading represent typical values or recommended default values.

Railroad:		Railroad Contact:	
Crossing DOT#:		Phone:	

Section 1: Right-of-Way Transfer Time Calculation					
Preempt Verification and Response Time					
1. Preempt delay time (seconds)		**0.0**	Usually 0.0 s		
2. Controller response time to preempt (seconds); get from manufacturer		**0.2**	Controller type:		
3. Preempt verification and response time (seconds)				0.2	
Worst-Case Conflicting Vehicle Time					
4. Worst-case conflicting vehicle phase number	**4**				
5. Minimum green time during right-of-way transfer (seconds)		**2.0**	2.0 s or more recommended		
6. Other green time during right-of-way transfer (seconds)		**0.0**	Usually 0.0 s		
7. Yellow change time (seconds)		**4.0**			
8. Red clearance time (seconds)		**1.0**			
9. Worst-case conflicting vehicle time (seconds)			7.0		
Worst-Case Conflicting Pedestrian Time					
10. Worst-case conflicting pedestrian phase number	**4**				
11. Minimum walk time during right-of-way transfer (seconds)		**2.0**	2.0 s or more recommended		
12. Pedestrian clearance time during right-of-way transfer (seconds)		**11.0**	o.k.		
13. Vehicle yellow change time (seconds)		**4.0**	Same as for normal operations.		
14. Vehicle red clearance time (seconds)		**1.0**	Same as for normal operations.		
15. Worst-case conflicting pedestrian time (seconds)			18.0		
Worst-Case Conflicting Vehicle or Pedestrian Time					
16. Worst-case conflicting vehicle or pedestrian time (seconds)				18.0	
17. Right-of-way transfer time (seconds)					**18**
Section 2: Queue Clearance Time Calculation					



Design Vehicle: Large School Bus

Approach Grade, %: 3% uphill

Warning Time Variability: Low

18. Clear storage distance, CSD (feet)	**25**				
19. Minimum track clearance distance, MTCD (feet)	**20**				
20. Design vehicle length, DVL (feet)	40				
21. Queue start-up distance, L (feet)		45			
22. Time required for design vehicle to start moving (seconds)			4.3		
23. Design vehicle clearance distance, DVCD (feet)		60			
24. Time for design vehicle to accelerate through the DVCD (seconds)			7.0		
25. Queue clearance time (seconds)				**11.3**	
Section 3: Maximum Preemption Time Calculation					
26. Right-of-way transfer time (seconds)		18.2			
27. Queue clearance time (seconds)		11.3			
28. Desired minimum separation time (seconds)		**4.0**	4.0 s recommended		
29. Maximum preemption time (seconds)				**33.5**	
Section 4: Sufficient Warning Time Check					
30. Required minimum time, MT (seconds); per regulations	**20.0**	20.0 s required by TMUTCD			
31. Clearance time, CT (seconds); get from railroad	**0.0**	AREMA requirement: 0.0 s			
32. Minimum warning time, MWT (seconds)		20.0	Excludes buffer time (BT)		
33. Advance preemption time, APT, if provided (seconds); get from railroad		**22.0**			
34. Warning time provided by the railroad (seconds)				42.0	
35. Additional warning time required from railroad (seconds)					**0**

Figure A-19. Preemption Worksheet Data Entry and Output.

REFERENCES

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