AN ITE PROPOSED RECOMMENDED PRACTICE

GUIDELINES FOR DETERMINING TRAFFIC SIGNAL CHANGE AND CLEARANCE INTERVALS

INSTITUTE OF TRANSPORTATION ENGINEERS

Guidelines for Determining Traffic Signal Change and Clearance Intervals

An ITE Proposed Recommended Practice

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ITE's purpose is to enable engineers and other professionals with knowledge and competence in transportation to contribute individually and collectively toward meeting human needs for safety and mobility by promoting professional development; supporting and encouraging education; stimulating research; developing public awareness; exchanging professional information; and by maintaining a central point of reference and action.

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Preface and ACKNOWLEDGMENTS

The Institute of Transportation Engineers (ITE) is preparing this report to reflect the current state-of-the-practice and to provide the user with a broad overview of key considerations to determine yellow change and red clearance intervals for traffic signals and their application. This report is being published as a proposed recommended practice of ITE. As such it is to be considered in its proposed form, but is subject to change after receipt and consideration of suggestions from those who have reviewed the report. Readers are encouraged to submit written suggestion(s) for improving this report to

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Written or e-mailed suggestions should be received at the above address no later than July 31, 2015 to ensure consideration for incorporation into the final recommended practice report.

ITE wishes to thank the members of each of the following committees and groups in their respective roles in the preparation of this report. This report was balloted and approved by a technical advisory committee of individuals involved in the development and review of the report. The voting members of the technical advisory committee are:

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(Letters in parentheses indicate ITE member grade: M-Member, F-Fellow)

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Chapter 1

INTRODUCTION

1.1 Background and Summary

The yellow change and red clearance intervals compose the two parts of the traffic signal change period. Divergent and strongly held positions characterize any discussion of this topic. Hundreds of papers and reports have been written on the subject by many authors from academia and the practicing profession. Even so, with the importance of the topic and the amount of study devoted to it, a consensus has been difficult to reach on the most appropriate method of timing the yellow change and red clearance intervals at traffic signals.

In 1985 ITE published a proposed recommended practice titled *Determining Vehicle Change Intervals*[1] that was not ratified by the ITE International Board of Direction to become a recommended practice. In 1994 ITE published an informational report prepared by the Technical Council Task Force 4TF-1 titled *Determining Vehicle Signal Change and Clearance Intervals*[2]. Later, in 2001, ITE published the informational report *A History of the Yellow and All-Red Intervals for Traffic Signals*.[3]

In the interim, changes in technology, automated enforcement, the availability of new primary data, further research, as well as the public and professional concern that a defined standard of reference did not exist with regard to this topic have led to the initiative to develop this report. ITE hosted a number of roundtable discussions at its Annual Meetings and Technical Conferences in recent years where the needs of public agencies have been clearly outlined. ITE is preparing this report to reflect the current state-of-the-practice and to provide the user with a broad overview of key considerations to determine yellow change and red clearance intervals for traffic signals and their application.

The guidelines are based not only upon existing information found during the initial research, but also on the collective experience of ITE staff, committee members, and the supporting consulting team. This report should not supersede engineering judgment. It is anticipated this document will be updated periodically to refine the procedures based on experiences of agencies using it and studies performed by the research community.

Note that this report is specifically focused on the timing of traffic signal change intervals. This report does not discuss or intend to discuss pedestrian signal change intervals nor methods of enforcement. ITE strongly supports appropriate application of engineering methods to time traffic signals.

1.2 Purpose and Intended Use

ITE's intent is, for the proposed recommended practice developed by this effort, to reflect a thoughtful balance between sound engineering theory and practical application. The underlying assumptions should yield reasonable times for the yellow change and red clearance intervals for traffic signals that allow the profession to balance those durations while enhancing intersection safety and maintaining reasonable traffic flow. The goal of the proposed recommended practice is to create a consensus methodology for calculating and evaluating traffic signal change intervals that can be uniformly and consistently implemented by transportation agencies.

The engineer must coordinate with designers who determine signal head treatments, and technicians who work with field assets to ensure calculated intervals are translated correctly into the actual yellow and red intervals displayed to road users on a signal face.

This recommended practice was written primarily for an audience of engineers engaged in the activity of determining yellow change and red clearance intervals. It is recognized that proper application of these intervals is dependent upon correct use of field equipment and engineering design applications. The engineer must coordinate with designers who determine signal head treatments, and technicians who work with field assets to ensure calculated intervals are translated correctly into the actual yellow and red intervals displayed to road users on a signal face.

Standards and recommended practices are used by consumers, manufacturers, public agencies, and suppliers to define their mutual obligations. They are essential for the orderly and efficient conduct of commerce and for the protection of the economic, social, environmental, and safety interests of all parties. Standards and recommended practices can favorably or unfavorably affect costs, availability, and performance of products and systems.

An important aspect of the development work of ITE is that all its standards and recommended practices are advisory only. ITE has no regulatory authority in which to enforce the use of these recommended practices. All standards and recommended practices are used and/or applied on substantially public facilities and only have status when officially sanctioned by the governing agency. Their use by public agencies is usually in the interest of safeguarding the welfare and safety of the private users of the products or facilities themselves. Public agencies are encouraged to use adopted ITE recommended practices and standards to support their local policies for the planning, design, management, maintenance, and operations of their traffic signal system. Significant benefit is derived by road users through the consistent design and application of traffic signal practices.

1.3 Sources of Information

Survey of Practice

For the purpose of this recommended practice, and in connection with the National Cooperative Highway Research Program (NCHRP) project that led to *NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections*,[4] a survey questionnaire was developed and distributed to a sample of national and international agencies. The survey was intended to identify differences and similarities in methods and factors used in traffic signal change interval practices. The survey was distributed in June 2009 to the following groups:

- Public agency members of the Traffic Engineering, Management and Operations/ITS, and Public Agency Councils of ITE;
- American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Traffic Engineering (state traffic engineers);
- A list of international organizations developed by the NCHRP research team; and
- A list of agency traffic engineers generated by ITE through the National Transportation Operations Coalition.

Ultimately, the questionnaire was disseminated to approximately 2,000 recipients. A copy of the questionnaire is included in Appendix A.

A total of 268 responses were received, 247 (92 percent) of which were from the United States and 20 (8 percent) of which were from Canada. One response was received from outside North America (Germany). Within the United States, responses were received from all 50 states except West Virginia.

Some general highlights and observations from the survey include the following:

- A majority of North American respondents (~60 percent) indicated their agency did not have a formal policy for timing traffic signal change intervals. This presents public agencies with potential issues in terms of inconsistent signal timing and tort liability.

- While there are various procedures across North America used in determining the duration of yellow change and red clearance intervals, engineering judgment plays a significant role.
- More than one-half of respondents use posted speed limits as a factor in the calculation of yellow change interval duration, compared with one-quarter that use 85th percentile approach speeds.
- Site-specific speed measurement data is generally updated infrequently.
- Agencies use a wide variety of procedures for special situations such as left- or right-turn signals, large trucks, pedestrians, and/or bicyclists.

Specific survey responses regarding methodology, parameters, and other factors are presented in the summary of the state of the practice in Chapter 2.

Outreach to the Profession

During the course of the project emphasis was placed on facilitating consensus on the subject matter through meetings and webinars on the following dates:

- ITE Technical Conference and Exhibit, Phoenix, AZ, USA, March 23, 2009
- Webinar 1, July 9, 2009
- ITE Annual Meeting and Exhibit, San Antonio, TX, USA, August 10, 2009
- Webinar 2, December 10, 2009
- Technical Advisory Committee Public Meeting at Transportation Research Board Annual Meeting, January 8, 2010
- ITE Technical Conference and Exhibit, Savannah, GA, USA, March 16, 2010
- ITE Annual Meeting and Exhibit, Atlanta, GA, USA, August 15, 2012
- ITE Technical Conference and Exhibit, San Diego, CA, USA, March 5, 2013

Comments from these sessions have been reviewed and, where appropriate, incorporated into the document presented here.

1.4 Definitions

The definitions presented in this document are from the 2009 *Manual on Uniform Traffic Control Devices* (MUTCD) with Revisions 1 and 2, except as noted.[5] The U.S. Department of Transportation Federal Highway Administration publishes the MUTCD and it is incorporated into the Code of Federal Regulations as the national standard for all traffic control devices installed on any street, highway, bikeway, or private road open to public. Key definitions from the MUTCD in this document include

- Interval—the part of a signal cycle during which signal indications do not change.
- Interval Sequence—the order of appearance of signal indications during successive intervals of a signal cycle.
- Red Clearance Interval—an (optional) interval that follows the yellow change interval and precedes the next conflicting green interval.
- Yellow Change Interval—the first interval following the green or flashing arrow interval during which the steady yellow signal indication is displayed.

A full glossary of terms is provided at the end of this report.

1.5 Related Projects

A separate effort by the NCHRP created *NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections*.[4] The objective of the NCHRP project was to develop a comprehensive and uniform set of recommended guidelines for determining safe and operationally efficient yellow change and red clearance intervals at signalized intersections based on the collection of new field data and in comparison to previous studies. The project conducted additional research to consider other factors that may be important in designing change intervals, including speeds, grades, vehicle types, vehicle mix, road surface conditions, sight distances, geometric considerations, coordinated systems and isolated signals, signal timing parameters, advanced detector locations, driver age, and turning movements. The reason for the new primary research was that the most recent studies of driver reaction times and vehicle deceleration rates used in determining appropriate yellow change and red clearance intervals were conducted more than 20 years ago. The NCHRP project included field studies on critical factors such as perception-reaction time, deceleration rates, start up delay, and the impact of the other factors identified as important in the design of change intervals.

1.6 Organization of the Report

This proposed recommended practice contains the following three primary chapters:

- **Chapter 1—Introduction:** provides an introduction to the project and provides the background on the subject of yellow and red intervals at traffic signals, as well as the context of the report in relation to other activities on the subject.
- **Chapter 2—State of the Practice:** describes the sources of methods and values presented in the proposed recommended practice. Each section provides a discussion of the relevant literature, current state of practice, comments received during the drafting process, and the recommendations used in the guidance chapter.
- **Chapter 3—Recommended Method of Determining Yellow Change and Red Clearance Intervals:** provides a description of the recommended methods to calculate traffic signal change intervals.

Chapter 2
STATE OF THE PRACTICE

2.1 Overview

The purpose of this chapter is to provide support for the methods and values presented in Chapter 3 defining the proposed recommended practice for timing yellow change and red clearance intervals for traffic signals. Each section will discuss relevant literature, current state of practice based on survey information as well as comments received during the drafting process, and the recommendations used in the guidance chapter.

2.2 Background

Fundamentally, the purpose of the circular steady yellow or solid yellow arrow signal indication is to warn vehicle traffic that the associated green movement is being terminated or that a red indication will be exhibited immediately thereafter.[6] Given the intent of the yellow indication to be a warning, drivers may enter the intersection until the red signal indication is displayed. A driver observing the yellow signal indication has two choices: 1) to come to a complete stop before entering the intersection, or 2) to proceed through the intersection, entering before the signal indication turns red. The intent of yellow change and red clearance intervals is to provide a safe transition between conflicting vehicular traffic movements. The goal of the engineering profession is to determine the ideal duration of yellow change and red clearance intervals that raises intersection safety while retaining a high level of operational efficiency.

The logic behind the methodology for determining the length of the yellow change interval is that the duration of the yellow change interval should provide a “reasonable” driver that is too close to the intersection to stop safely and comfortably with adequate time to traverse the distance to and legally enter the intersection before the signal turns red or right of way terminates. The yellow signal indication is not meant to cover the time to comfortably stop inasmuch as part of the stopping maneuver can safely occur during the red signal indication. A “reasonable” driver closer to the intersection will proceed into the intersection when presented with a yellow indication. A “reasonable” driver farther away from the intersection at the onset of the yellow indication will decide to stop and has sufficient distance to do so comfortably.

2.3 Calculation Method

A review of relevant literature identified multiple methods for determining traffic signal yellow change and red clearance intervals:†

- Kinematic equation method;
- Rule-of-thumb method;
- Uniform value method;
- Stopping probability method;
- Combined kinematic model and stopping probability method;
- Modified kinematic model for left-turn movements;
- Conflict zone method; and
- Rational models method.

Literature

Kinematic Equation Method

The kinematic equation method is the most widely known and recognized method for determining yellow change intervals. This method establishes the yellow change interval as the combination of the perception-reaction time (PRT)†† and the time to traverse the braking distance to the intersection, and the red clearance interval as the time to travel through the intersection.

The method begins with the following kinematic equation:

$$v_f^2 = v_o^2 + 2ad \tag{1}$$

Where:

v_o = original velocity (ft./sec.);
v_f = final velocity (ft./sec.);
a = constant acceleration (or deceleration) (ft./sec./sec.); and
d = distance (ft.).

For a body in motion to come to rest, v_f is set to a value of 0.

$$0 = v_o^2 + 2ad \tag{2}$$

Rearranging and assuming a is a negative value as deceleration changes the equation to

$$d = v_o^2/2a \tag{3}$$

In the foundational paper by Gazis, Herman, and Maradudin[7] for the condition of a driver coming to a complete stop before entering the intersection, the authors define the critical distance as

$$x_c = v_o\delta_2 + v_o^2/2a_2^* \tag{4}$$

Where:

x_c = critical distance (ft.);
v_o = original velocity (ft./sec.);
δ_2 = reaction time (sec.); and
a_2^* = constant deceleration, noted as maximum safe and comfortable rate (ft./sec./sec.).

They state that if the distance from the intersection x is greater than x_c then the vehicle can be stopped, and that if x is less than then x_c it may be uncomfortable or unsafe to stop. This critical distance is independent of the yellow change interval duration.

By noting that when x_o corresponds to x_c, a minimum yellow indication duration is calculated for the change and clearance time of an approaching vehicle:

$$\tau_{min} = \delta_2 + \frac{1}{2}v_o/a_2^* + (w + L)/v_o \tag{5}$$

Where:

τ_{min} = total change period (sec.);
x_c = critical distance (ft.);
v_o = original velocity (ft./sec.);
δ_2 = reaction time (sec.);
a_2^* = constant deceleration, noted as maximum safe and comfortable rate (ft./sec./sec.);
w = effective width of intersection (ft.); and
L = length of the vehicle (ft.).

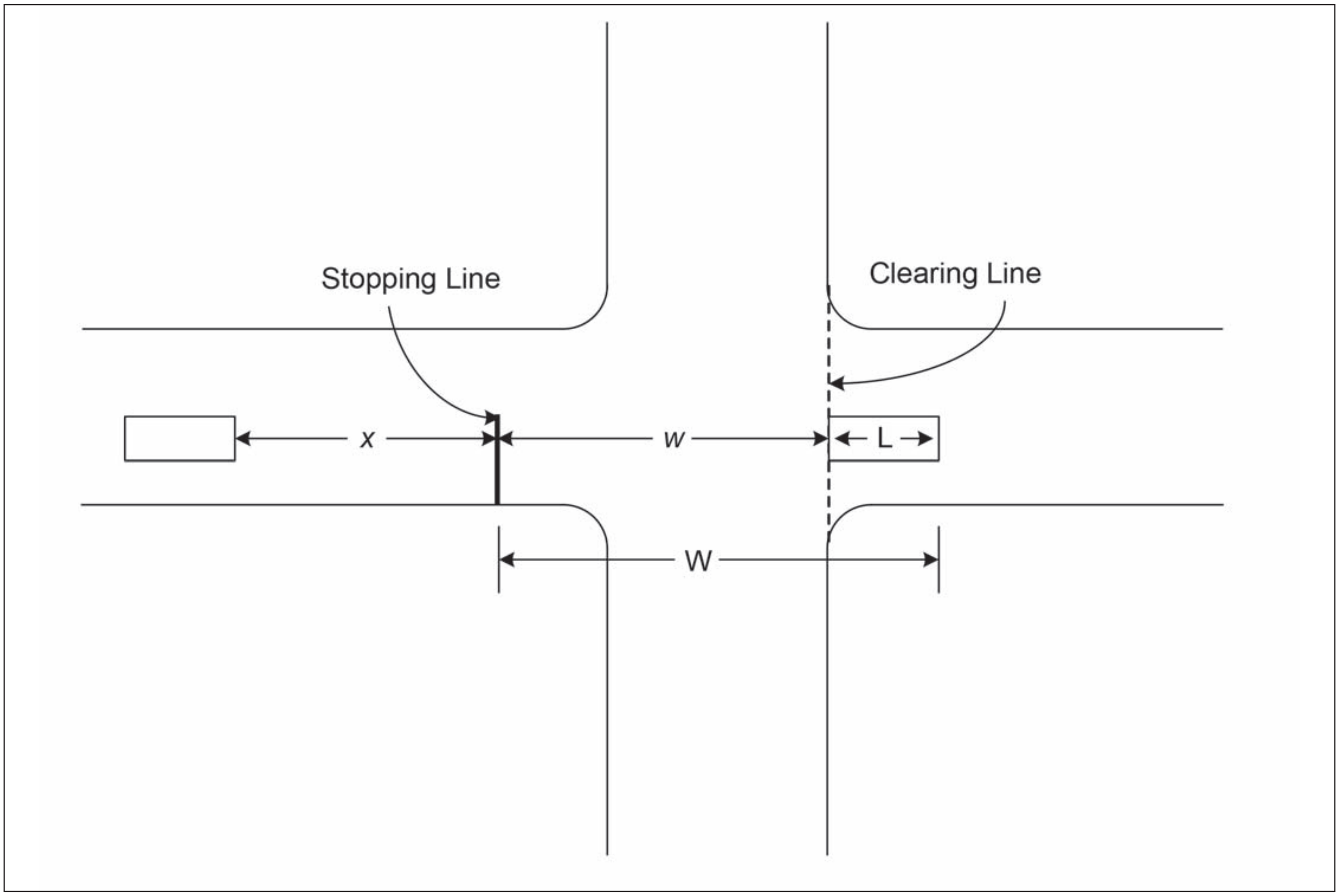
The application, as noted by the authors, in the original derivation is for the through movement at a single traffic signal, although they mention that results can be obtained using analogous methods for closely spaced signals at a divided highway or other variations such as turning movements.

All methods proposed by ITE for determining yellow change and red clearance intervals since 1965 have been based on the kinematic equation method. *A History of the Yellow and All-Red Intervals for Traffic Signals*[3] provides a comprehensive review of these models and formative research prior to the kinematic model. The report indicates the standard kinematic model has had few changes since its adoption in 1965. A modification factor to accommodate approach grade was incorporated in 1982 and has since been in the equation(s). The *ITE Traffic Engineering Handbook,* 6th Edition[8] provides Equation 6 and Equation 7 for calculating the yellow change and red clearance interval, or change period:

† Please note that equations in this section are provided in the units as noted in the original reference.

†† The term is commonly referred to in literature as "perception-reaction time." This time interval consists of the perception time of the driver and the reaction time of the driver-vehicle system is composed of the driver depressing the brake pedal and the vehicle reacting to this input by applying the brakes.

Figure 2.1: Intersection Geometry for Variables in Gazis, Herman, and Maradudin Derivation



$$Y = t + \frac{v}{2a + 2Gg} \tag{6}$$

$$R = \frac{W + L}{v} \tag{7}$$

Where:

- Y = yellow change interval (sec.);
- t = perception-reaction time (typically 1 sec.);
- v = design speed (ft./sec.);
- a = deceleration rate (typically 10 ft./sec./sec.);
- G = acceleration due to gravity (32.2 ft./sec./sec.);
- g = grade of approach (percent/100, downhill is negative grade);
- R = red clearance interval (sec.);
- W = width of intersection, stop line to far side no-conflict point (ft.); and
- L = length of vehicle (typically 20 ft.).

The earlier *Traffic Engineering Handbook,* 4th Edition[9] and *Determining Vehicle Change Intervals: A Proposed Recommended Practice*[1] also advocated calculating the red clearance interval separately using the third term of the kinematic equations.

This method accommodates varying pedestrian conditions as shown in Equations 8, 9, and 10.

$$r = \frac{w + L}{v} \tag{8}$$

$$r = \frac{P}{v} \tag{9}$$

$$r = \frac{P + L}{v} \tag{10}$$

Where:

- r = length of the red clearance interval (sec.);
- w = width of the intersection (ft.), measured from the near side stop line to the far edge of the conflicting traffic lane along the actual vehicle path;
- P = width of intersection (ft.) measured from the near side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path;
- L = length of vehicle, recommended as 20 ft.; and
- v = speed of the vehicle through the intersection (ft./sec).

The subsequent 1999 edition of the *Traffic Engineering Handbook,* 5th Edition,[10] however, reverts to the pre-1985 guidance and recommends the choice whether to use a red clearance interval be determined by intersection geometries, collision experience, pedestrian activity, approach speeds, local practices, and engineering judgment.

Section 4D.26 of the 2009 *Manual on Uniform Traffic Control Devices* (MUTCD)[5] provides the following support for determining change intervals based on engineering practices:

> *"Engineering practices for determining the duration of yellow change and red clearance intervals can be found in ITE's "Traffic Control Devices Handbook" and in ITE's "Manual of Traffic Signal Design" (see Section 1A.11)."*

Both referenced publications refer to the kinematic equation method for determining change intervals. The Federal Highway Administration (FHWA) *Traffic Signal Timing Manual*,[11] published in 2008, also suggests applying the kinematic equation method.

NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections [4] reviews the background, literature, and collected new data for intersection parameters at 83 sites around the country and 7,482 vehicles. The authors state the kinematic method is the preferred method and evaluated the data as it relates to the values of the various equation parameters and recommended values. The authors formulated the red clearance interval with a 1 sec. subtraction for conflicting approach start-up delay. The report provides the following equations and associated parameters for calculating the yellow change and red clearance intervals:

$$Y = t + \frac{1.47V}{2a + 64.4g} \tag{11}$$

$$R = \frac{W + L}{1.47V} - 1 \tag{12}$$

Where:

Y = yellow change interval (sec.);
t = perception-reaction time (set at 1.0 sec.);
V = 85th percentile approach speed (mph);
a = deceleration rate (typically 10 ft./sec./sec.);
g = grade of approach (percent/100, downhill is negative grade);
R = red clearance interval (sec.);
W = width of intersection, stop line to far side no-conflict point (ft.); and
L = length of vehicle (set at 20 ft.).

The ITE *Traffic Control Devices Handbook*, 2nd Edition[12] provides two forms of the equations for the yellow change and red clearance intervals based on the kinematic equation converted to enter speed in mph and adapts both versions of the equation to metric units in km/h. The document notes the guidance from the NCHRP report and provides information on the 1 sec. start-up delay subtraction from the red clearance interval.

Rule-of-Thumb Method

The rule-of-thumb method was cited in the 1994 ITE Technical Council Task Force 4TF-1 report, *Determining Vehicle Signal Change and Clearance Intervals*,[2] as a method used by practitioners for determining yellow change intervals. The method calls for calculating the yellow change interval by dividing the approach speed in miles per hour by 10 (Equation 13). Typically the 85th percentile approach speed or posted speed is used.

$$Y = \frac{V}{10} \tag{13}$$

Where:

Y = yellow change interval (sec.); and
V = approach speed (mph).

Uniform Value Method

The uniform value method applies a single yellow change or red clearance interval to all intersections in a jurisdiction or along an arterial. The practitioner typically determines an appropriate yellow change or red clearance interval for local conditions based on engineering judgment. *A History of the Yellow and All-Red Intervals for Traffic Signals* referenced Benioff et al.,[13] Frantzekakis,[14] and Wortman et al.[15] in discussing uniform change intervals. The before-after study by Benioff et al. did not demonstrate an increase in safety after implementing a somewhat uniform yellow change interval of 3.4 to 4.0 sec. in the Fresno/Clovis, CA metropolitan area. However, Frantzekakis supported constant yellow change intervals based on approach speeds to help prevent driver confusion. Wortman et al. also advocated a uniform yellow change interval of 4 sec. based on findings that driver behavior is dependent on deceleration and independent of intersection conditions.

Stopping Probability Method

As referenced in *A History of the Yellow and All-Red Intervals for Traffic Signals*, Olson and Rothery[16] developed a method for determining the yellow change interval based on the driver's probability of stopping as a function of the distance to the intersection. Driver stopping behavior observations indicated behavior did not change significantly with longer yellow change intervals. They found the 95th percentile distance back from the intersection at which the vehicles would still stop for the yellow indication. The researchers concluded the yellow change interval can be modeled based on driver behavior (Equation 14).

$$\tau_{\min} = \frac{A + W + L}{V_o} \tag{14}$$

Where:

$\tau_{\min}$ = yellow change interval (sec.);

A = distance to intersection (ft.), where 95 percent of vehicles will stop for the yellow indication (based on probability curves);

W = width of the intersection (ft.);

L = length of the vehicle (ft.), recommended as 20 ft.; and

V_o = speed of approaching vehicles (ft./sec.).

Combined Kinematic Model and Stopping Probability Method

A History of the Yellow and All-Red Intervals for Traffic Signals references a 1977 study by Williams[17] that proposed a combined kinematic model and stopping probability method. Based on stopping behavior observations at one intersection, Williams developed a model for determining the yellow change interval as a function of intersection conditions and a probability curve based on deceleration rate observations of the driver and acceleration rate of the conflicting vehicle (see Equation 15).

$$Y = R + \frac{V}{2a^-} + \frac{(W + L)}{V} - \left[K + \sqrt{\frac{2d}{a^+}}\,\right] \tag{15}$$

Where:

Y = yellow change interval (sec.);

R = driver decision and reaction time (1.1 sec.);

V = 85th percentile approach speed (m/sec. [ft./sec.]);

a^- = deceleration accepted 85 percent of the time (2.0 m/sec./sec. [6.5 ft./sec./sec.]);

W = distance from stop line to the line where the vehicle is shadowed;

L = length of vehicle (5 m [17 ft.] for automobiles);

K = reaction time of cross-flow traffic (0.4 sec.);

d = distance between vehicles and cross-flow traffic (m [ft.]); and

a^+ = maximum acceleration of cross-flow traffic (4.9 m/sec./sec. [16 ft./sec./sec.]).

Modified Kinematic Model for Left-Turn Movements

Liu, Yu, Saksit, and Oey[18] modified the kinematic model to accommodate the entering speed of a left-turning vehicle. The researchers developed the proposed model to account for deceleration or acceleration as a vehicle makes a left turn. The resulting model for determining the yellow change interval for left-turn movements is shown in Equation 16.

$$y_t = 2\frac{\left(\delta\_ + \dfrac{v_l}{2a\_}\right)}{\left(1 + \dfrac{v_i}{v_l}\right)} \tag{16}$$

Where:

y_t = yellow change interval for left-turn movements (sec.);

δ = perception-reaction time for decelerating vehicle (sec.);

v_l = speed limit along approaching direction before making left turn (m/sec.);

$a\_$ = comfortable deceleration rate when approaching intersection (m/sec./sec.); and

v_i = vehicle speed when entering intersection for making left turn (m/sec.).

Liu et al. compared the calculated yellow change intervals with existing intervals at two intersections in Texas. Observations indicated the calculated interval was appropriate for one intersection, but may not have been sufficient for the second intersection. The study also concluded yellow change intervals should generally be longer for left-turning movements than those for the straight-through movements on the same approach.

> ***Observations by Liv, et al. indicated the calculated interval was appropriate for one intersection, but may not have been sufficient for the second intersection.***

Yu, Qiao, Zhang, Tian, and Chaudhary[19] built upon the study by Liu et al. by further modifying the kinematic equation to accommodate a delay due to low visibility of the traffic signal. The addition of the delay term resulted in Equation 17 for determining the yellow change interval for left-turn movements.

$$y_t = 2\frac{\left(\delta\_ + \dfrac{v_l}{2a\_}\right)}{\left(1 + \dfrac{v_i}{v_l}\right)} + T_{vi} \tag{17}$$

Where:

y_t = yellow change interval for left-turn movements (sec.);

$\delta\_$ = perception-reaction time for decelerating vehicle (sec.);

v_l = speed limit along approaching direction before making left turn (m/sec.);

$a\_$ = comfortable deceleration rate when approaching intersection (m/sec./sec.);

v_i = vehicle speed when entering intersection for making left turn (m/sec.); and

T_{vi} = delay due to low signal visibility (sec.).

Yu et al. collected data and performed field observations at 21 intersections and concluded the calculated yellow change intervals for protected left-turn movements were generally shorter than the existing intervals. This study also proposed a red clearance interval calculation method for protected left-turn movements which resulted in longer calculated intervals than existing intervals. The red clearance interval method is further discussed in the subsequent section. Yu et al. concluded that the calculated total change intervals were similar to existing change intervals. The research for this proposed method for determining change intervals for left-turn movements was sponsored by the Texas Department of Transportation for a guidebook in 2004.[20]

> ***Yu et al. concluded the calculated yellow change intervals for protected left-turn movements were generally shorter than the existing intervals.***

In addition to proposing a yellow change interval model for left-turn movements, Liu, Yu, Saksit, and Oey also modified the kinematic model for the red clearance interval for left-turning vehicles. The model, shown in Equation 18, uses the same kinematic equation form of distance divided by speed, but the researchers added variables to the equation.

$$\tau_t = \frac{S}{\bar{v}_c} \tag{18}$$

Where:

τ_t = red clearance interval (sec.);

S = length of the curve measured from the stop line to one vehicle length ahead of the clearance line (m); and

$\bar{v}_c$ = average speed of the left-turning vehicle, in m/sec.

The length of the curve is estimated as a value between a minimum and maximum curve distance (see Equation 19).

$$S = \beta S_{max} + (1 - \beta) S_{min} \tag{19}$$

Where:

S = length of the curve measured from the stop line to one vehicle length ahead of the clearance line (m);

S_{max} = $w_t + L + w_l$ (m);

S_{min} = $\sqrt{\tilde{w}_t^2 + w_l^2 + 2\tilde{w}_t w_l \cos\phi}$ (m); and

$\beta \in (0,1)$ dimensionless parameter ranges from 0 to 1 of estimating turning length S.

The average speed of the left-turning vehicle is determined by Equation 20.

$$\bar{v}_c = \min\{[\sqrt{\frac{\gamma g S}{\phi}}, \theta v_l + (1-\theta) v_{lt}\} \tag{20}$$

Where:

$\bar{v}_c$ = average speed along the length of the curve measured from the stop line to one vehicle length ahead of the clearance line (m/sec.);

γ = dimensionless parameter for estimating magnitude of acceleration rate that left-turning driver is willing to bear (ranges from 0.3 to 0.8);

g = gravitational rate of acceleration (m/sec./sec.);

S = length of the curve measured from the stop line to one vehicle length ahead of the clearance line (m);

ϕ = intersecting angle between vehicle approach and departure direction (radians);

θ = dimensionless parameter estimating turning speed (ranges from 0 to 1);

v_l = speed limit along approaching direction before curve (m/sec.); and

v_{lt} = speed limit along departure direction when curve ends (m/sec.).

As with the proposed yellow change interval method, Liu et al. compared the calculated red clearance intervals with existing intervals at two intersections in Texas. Existing red clearance intervals were 1 sec., which the researchers determined was insufficient for the observed clearance times. Varying the acceleration parameter, the researchers noted that calculated red clearance times were approximately 3 sec. or more for both intersections. The researchers provided several conditions in which shorter red clearance intervals may be successful. These included cases in which the last left-turning vehicle clears the intersection before the yellow indication ends or maneuvers the curve quickly during the red clearance interval or when the conflicting vehicle yields to a left-turning vehicle after the red clearance interval. Liu et al., however, also noted that during inclement weather or low visibility conditions, these shorter red clearance intervals may be dangerous.

> ***Yu et al. further modified the proposed method by incorporating the correction factor, ξ, which considers the curve of a left-turning vehicle on an approach with multiple left-turn lanes and/ or a vehicle turning onto a multi-lane road.***

Yu et al. further modified the proposed method by incorporating parameters for the number of lanes on approaching and conflicting lanes and the distance between potential conflict points and stop lines. The red clearance interval is calculated by Equation 21.

$$r = 0.6820\frac{\xi(S + L)}{V_c} - T_{cs} \quad (21)$$

Where:

r = red clearance interval (sec.);

ξ = correction factor for number of lanes;

S = length of the curve measured from the stop line to one vehicle length ahead of the clearance line (m);

L = vehicle length (m);

V_c = average vehicle speed (m/sec.); and

T_{cs} = time deduction for distance between potential conflict points and stop lines (sec.).

The correction factor, ξ, considers the curve of a left-turning vehicle on an approach with multiple left-turn lanes and/or a vehicle turning onto a multi-lane road. The additional time deduction, T_{cs}, subtracts the time required for vehicles to reach the conflict point. This deduction accommodates the left-turning vehicle as it moves from the stop line to the conflict point or the conflicting vehicle that starts from a stop position and accelerates to the conflict point. Additional information on these variables is published in Yu et al.

Yu et al. calibrated the modified method with 21 intersections in Texas. Results indicated the mean of the calculated red clearance intervals was longer than the mean of the existing timings, but shorter than the mean of the actual, observed vehicle clearance time. The findings suggested the existing red clearance intervals were insufficient. The study also addressed the proposed yellow change interval calculation method, which resulted in calculated timings that were shorter than existing timings. Yu et al. concluded that since overall change intervals were similar to calculated change intervals, the change interval timings could be modified without negatively affecting intersection efficiency.

Conflict Zone Method

Muller, Dijker, and Furth[21] proposed a method for determining the red clearance interval based on the distance between the entering and exiting streams of conflicting traffic. The "conflict zone" refers to the area in the intersection in which the conflicting traffic paths first overlap. The required red clearance interval is calculated as Equation 22.

$$R = t_{exit} - t_{entrance} \quad (22)$$

Where:

R = red clearance interval (sec.);

t_{exit} = time required for exiting stream vehicle to travel from the stop line to the end of the conflict zone (sec.); and

$t_{entrance}$ = time required for entering stream vehicle to reach the conflict zone (sec.).

The exit time, t_{exit}, is determined for the last vehicle that crosses the stop line at the end of the yellow indication (see Equation 23).

$$t_{exit} = \frac{s_{exit}}{v_{exit}} \quad (23)$$

Where:

t_{exit} = time required for exiting stream vehicle to travel from the stop line to the end of the conflict zone (sec.);

s_{exit} = distance traveled by the exiting stream vehicle from the stop line to the end of the conflict zone, including vehicle length of 12 m; and

v_{exit} = speed of exiting stream vehicle (m/sec.).

The entrance time, $t_{entrance}$, reflects the conflicting vehicle entering the intersection after receiving the green indication. This time is determined for the vehicle with the lowest entering time. Equation 24 considers an entering vehicle that begins from the stop position and a vehicle that accelerates from an approaching speed.

$$t_{entrance} = \begin{cases} t_r + \sqrt{\dfrac{2 \times s_{entrance}}{a_{acc} - a_{dec}}} & , \; s_{entrance} \leq s_{critical} \\ t_r + \dfrac{s_{entrance}}{v_{\max}} + \dfrac{v_{\max}}{2 \times (a_{acc} - a_{dec})} & , \; s_{entrance} > s_{critical} \end{cases} \quad (24)$$

Where:

$t_{entrance}$ = time required for entering vehicle to reach the conflict zone (sec.);

t_r = reaction time (sec.);

$s_{entrance}$ = distance traveled by the entering vehicle to the stop line (m);

$s_{critical}$ = distance at which the minimum entrance time is valid (m);

a_{dec} = constant deceleration approaching intersection (m/sec./sec.);

a_{acc} = constant acceleration after green indication (m/sec./sec.); and

v_{max} = maximum approach speed of entering vehicle (m/sec.).

Muller, Dijker, and Furth calibrated the conflict zone method for two intersections in the Netherlands. The resulting red clearance intervals were shorter than those calculated from the ITE equation, suggesting this method produces more efficient durations. The conflict zone method was published in Dutch traffic signal guidelines in 1996.

Rational Models Method

Fitch, Shafizadeh, Zhao, and Crowl[22] also proposed a simplified version of the conflict zone method in a presentation at the ITE 2008 Technical Conference and Exhibit. They reported on a method for determining yellow change and red clearance intervals based on rational models. The method was implemented by the Sacramento [CA, USA] County Department of Transportation in 1998. A report summarizing the results of a before-and-after study of the implementation is currently being drafted for publication.

Fitch et al. suggested calculating the yellow change interval based on the time for a vehicle traveling at the 90th percentile speed to travel from the far dilemma zone boundary at the 10th percentile stopping distance to the stop line.

The proposed method for determining the red clearance interval is shown in Equation 25.

$$R = t_c - t_{min} \tag{25}$$

Where:

R = red clearance interval (sec.);

t_c = time for vehicle receiving yellow indication to clear conflict point (sec.); and

t_{min} = minimum time for vehicle receiving green indication to arrive at conflict (sec.).

The minimum time for a vehicle to accelerate to the conflict point, t_{min}, is shown in Equation 26:

$$t_{min} = \sqrt{\frac{2D}{a_s - a_r}} \tag{26}$$

Where:

t_{min} = minimum time for vehicle receiving green indication to arrive at conflict point (sec.);

D = distance to the conflict point beyond the stop bar (ft.);

a_s = acceleration of vehicle after onset of green (ft./sec./sec.); and

a_r = deceleration of vehicle prior to onset of green (ft./sec./sec.).

Based on an average deceleration rate of 10 (ft./sec./sec.) and an acceleration rate of 15 (ft./sec./sec.), as suggested by the researchers, Equation 27 becomes

$$t_{min} = 0283\sqrt{D} \tag{27}$$

Where:

t_{min} = minimum time for vehicle receiving green indication to arrive at conflict point (sec.); and

D = distance to the conflict point beyond the stop bar (ft.).

Current Practice

A majority of respondents to the survey, 161 of 267 (60%), indicated their agency did not have a formal policy for timing the yellow change interval. No respondents indicated they did not know the answer to this question, suggesting a clear understanding of agency policies on this issue. A somewhat greater number of respondents, 167 of 267 (63%), indicated their agency did not have a formal policy for timing the red clearance interval. Again, no respondents indicated they did not know the answer to this question.

Respondents were asked what method they generally used to determine the duration of change intervals in the absence of a formal agency policy. A total of 217 responses to this question were received. This was a multiple-choice question, with questions and responses summarized in Table 2.1.

Table 2.1: General Methods Used to Determine Duration of Yellow Change Intervals in the Absence of a Formal Agency Policy

Method	no. of responses	%
The kinematic equation: CP = t + V / (2a+64.4g) + (W+L) / V	85	39
Uniform value for all intersections	12	6
Uniform value for all intersections, except where conditions warrant an exception	42	19
A table of values by approach speed applied to all intersections	38	18
Other	40	18
Total	**217**	**100**

Respondents who answered "other" to the previous question regarding methods to determine duration of change intervals in the absence of a formal agency policy were given the option to provide information on alternate formulae, alternate policies, or other methods used in practice. Information regarding these other methods is summarized in Table 2.2.

Table 2.2: Other Methods Used to Determine Duration of Yellow Change Intervals in the Absence of a Formal Agency Policy

Alternate Formula	Alternate Policy	Other
3.5 to 4 sec. on low speed approaches and 5+ on other	California Supplement MUTCD Table 4D-102 (Yellows)	Evaluate each intersection individually
4 sec. - major approach 3 sec. - minor approach	Caltrans Policy	Uniform Yellow + Calculated Red Clearance Interval
if over 35 mph, 4 sec. if under, 3 sec.	Caltrans recommended intervals	Formal policy based on the ITE formula
t +V/(2a+64.4g) for yellow, min 3.5 sec.	Follow guidelines in the California MUTCD	Kinematic arterials + some side streets, rest uniform
V= posted speed + 5 for Y	Guidelines outlined in the California MUTCD	Meets or exceed CA MUTCD minimum
Y = mph/10 in sec.	British Columbia Ministry of Transportation—Electrical and Traffic Engineering Manual	Yellow is done from a table by approach speed
Yellow = t+V/(2a+64.4g), Red = 1.0 to 2.0 sec.	Connecticut DOT guidelines	A table of values by approach speed plus condition
yellow 4 sec. for 50 km/h, 5 sec., for 60 mph or faster	Idaho DOT Policy	Based on approach speed and engineering judgment
CP = t + V / 2a	TRB 1992 signal timing improvement practices	For all-red, engineering judgment is used
Separate formula-based tables	ITE Proposed Recommended Practice for Clearance Intervals (1985)	Kinematic equation with rounding
Yellow = kinematic equation	Follow ITE formula for change intervals	Kinematic equation compare with uniform values

Recommendation

As part of the review process, the study team reviewed relevant research on different methods of calculating traffic signal change intervals and welcomed suggestions for alternative methods from the transportation community. Currently, sufficient evidence does not exist in the transportation engineering community to support recommending some of these methods for widespread use because of inadequate experience in practice, lack of documentation of significant safety benefits, and/or limited practicality for field implementation. At this time, alternative methods described in the literature are not preferred due to the limited body of supporting research and varying acceptance of alternative methods by the transportation community. Although some methods have been studied in select locations, sufficient evidence does not exist to support that these methods are improvements over methods based on the kinematic equation.

Based on an evaluation of the available information, the method based on the kinematic equation is preferred for determining the yellow change and red clearance intervals as currently formulated in the most recent research on the subject. An evaluation of the state-of-the-practice indicated a majority of agencies in the United States currently use some form of the kinematic equation. This method is one of the more widely recognized and accepted methods in the traffic engineering community. Other methods also currently applied in the field are the uniform value and rule-of-thumb methods. Based on the survey results, the transportation community prefers the kinematic equation method over these latter two methods for most cases because it is more adaptable to various conditions. The limitations of the kinematic equation method are that it 1) assumes uniform deceleration, which may be an oversimplification of driver behavior, and 2) assumes a vehicle that does not stop for the yellow proceeds to and across the intersection at a constant speed equal to its approach speed. This method also assumes the potential conflict area can be defined by the intersection width, which may not accurately represent the actual conflict zone. The strength of this method is that change intervals are calculated based on equal critical distances for stopping or proceeding through the intersection based on a comfortable deceleration rate.

The strength of the kinematic equation method is that change intervals are calculated based on equal critical distances for stopping or proceeding through the intersection based on a comfortable deceleration rate.

This kinematic method is the basis for the rest of the discussions in this proposed recommended practice. The equations are noted below:

$$Y = t + \frac{1.47V}{2a + 64.4g} \quad (\textit{U.S. units}) \ (28)$$

$$R = \left[\frac{W + L}{1.47V}\right] - t_s \quad (\textit{U.S. units}) \ (29)$$

Where:

- Y = yellow change interval (sec.);
- t = perception-reaction time (sec.);
- V = 85th percentile approach speed (mph);
- a = deceleration rate (ft./sec./sec.);
- g = grade of approach (percent/100, downhill is negative grade);
- R = red clearance interval (sec.);
- W = width of intersection, stop line to far side no-conflict point (ft.);
- L = length of vehicle (ft.); and
- t_s = conflicting movement start up delay (sec.).

$$Y = t + \frac{0.28V}{2a + 19.6g} \quad (\textit{Metric units}) \ (30)$$

$$R = \left[\frac{W + L}{0.28V}\right] - t_s \quad (\textit{Metric units}) \ (31)$$

Where:

- Y = yellow change interval (sec.);
- t = perception-reaction time (sec.);
- V = 85th percentile approach speed (km/h);
- a = deceleration rate (m/sec./sec.);
- g = grade of approach (percent/100, downhill is negative grade);
- R = red clearance interval (sec.);
- W = width of intersection, stop line to far side no-conflict point (m);
- L = length of vehicle (m); and
- t_s = conflicting movement start up delay (sec.).

This formulation of the equations comes from the most recent research document, *NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections*.[4] Consideration is given to the conflicting approach start-up delay as a factor in the red clearance interval equation with notation for conflicting traffic noted as t_s rather than simply a subtraction of a value of "1." Additionally, the equations are provided in a formulation that allows input to the equation of speed in mph (km/h) rather than ft./sec.

2.4 Variance in Vehicle Codes

Literature

Each state has enacted statutes governing entry of vehicles into the intersection during the change interval. There are two generally recognized legal principles for the meaning of change intervals—the *permissive* law and the *restrictive* law.

Under *permissive* laws, drivers may enter the intersection during the yellow interval and legally be in the intersection while the red signal indication is displayed, as long as the driver entered before or during the yellow signal indication. Jurisdictions with *permissive* laws may use a red clearance interval to ensure drivers can clear the intersection prior to the change in right-of-way even though traffic conflicting with the vehicles clearing the intersection is required to yield to other vehicles and pedestrians lawfully within the intersection.

Under *restrictive* laws, drivers may not enter the intersection during the yellow signal indication unless the intersection can be cleared prior to onset of the red indication or unless it is impossible or unsafe to stop.

The 2009 MUTCD as revised[5] states,

> *"Vehicular traffic facing a steady CIRCULAR YELLOW signal indication is thereby warned that the related green movement or the related flashing arrow movement is being terminated or that a steady red signal indication will be displayed immediately thereafter when vehicular traffic shall not enter the intersection. The rules set forth concerning vehicular operation under the movement(s) being terminated shall continue to apply while the steady CIRCULAR YELLOW signal indication is displayed."*

According to Section 4D.04 of the MUTCD, drivers receiving the subsequent green indication and pedestrians receiving the walk indication are to yield right-of-way to vehicles legally in the intersection before proceeding.

The *Uniform Vehicle Code 2000* (UVC)[6] states, "Steady yellow indication: vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter."

Current Practice

Forty-six U.S. states and 12 Canadian provinces have statutes in substantial conformity with the meaning of the yellow and red indications in the MUTCD and UVC. Nine of these jurisdictions permit motorists to drive cautiously through the intersection on the red only if too close to stop safely. Four states—Louisiana, Tennessee, Rhode Island, and West Virginia—prohibit vehicles from crossing or being in the intersection on red. The statutes in these four states conflict with the MUTCD. Appendix B provides the definition of vehicular yellow signal indication by state.

Recommendation

Variation in motor vehicle statutes has a large effect on methodologies used for calculating the timing and use of the yellow change and red clearance intervals. The large majority of jurisdictions have motor vehicle statutes with *permissive* laws which agree with the requirements of MUTCD. Therefore, the recommendations of this recommended practice use an approach conforming to the MUTCD with values applicable to agencies in jurisdictions with *permissive* statutes that allow vehicles to enter on yellow and be in the intersection on red as long as the vehicle entered on yellow.

2.5 Perception-Reaction Time

Literature

A History of the Yellow and All-Red Intervals for Traffic Signals[3] summarizes the history of the PRT variable. The report states a 1.1 sec. PRT was used implicitly in the 1941 and 1950 edition of the ITE *Traffic Engineering Handbook,* 2nd Edition.[23]

The 1960 Gazis et al.[7] observational study of 87 drivers found a mean PRT of 1.14 sec., with a range from 0.6 to 2.4 sec. The findings appeared to support the 1.1 second PRT over the 1 sec. value later suggested by ITE in the 1965 edition of the *Traffic Engineering Handbook*.[24] All subsequent editions of the *Traffic Engineering Handbook* have suggested this 1 sec. PRT.

In 1977, Williams[17] suggested applying the earlier 1.1 sec. PRT in the combined kinematic model and stopping probability method, although he also noted the importance of engineering judgment in calculating change intervals.

Chang, Messer, and Santiago[25] reported similar PRT values in a 1985 observational study of 579 drivers at 13 intersections with approach speed limits of 30 to 55 mph (50 to 90 km/h). The researchers reported a mean PRT of 1.3 sec. and a range of 0.7 to 1.55 sec. Their findings also supported a relationship between PRT and the distance to the intersection, approach speed, and time available to reach the stop line. Based on the findings, Chang et al. recommended a PRT of 1.2 sec.

The *Handbook for Designing Roadways for the Aging Population*[26] references Tarawneh's 1991 thesis from the University of Nebraska, which supported a 1.5 sec. PRT.[27] The work reviewed the history of PRT and examined human factors affecting older-driver PRT.

A 1995 experimental study by Knoblauch et al.,[28] which reported PRT values lower than 1 sec., was also cited in the *Handbook for Designing Roadways for the Aging Population* for research on age-related PRT. The study observed 81 drivers at a low-speed intersection in a controlled testing facility. As part of a larger study on older-driver behavior, this study examined PRT for drivers aged 60 and older as well as drivers younger than 60. Drivers approached the test intersection at 20 and 30 mph (32.2 and 48.3 km/h), and the yellow indication was displayed when time from the traffic signal was approximately 3.5 and 4.5 sec.. When distance to the intersection was longer, such as during higher approach speed data points, the 85th percentile older-driver PRTs were 1.38 sec. and 0.88 sec., significantly longer than the younger-driver PRTs of 0.50 sec. and 0.46 sec. The researchers attributed this difference to older drivers taking additional time to respond when there is more available time to traverse the braking distance. The findings also did not support a significant difference between the 85th percentile PRT for the older- and younger-aged driver groups when distance to the intersection was shorter. The researchers concluded that change interval calculation methods did not need to be modified to accommodate older drivers.

Based on the conflicting findings by Tarawneh and Knoblauch, the *Handbook for Designing Roadways for the Aging Population* concluded the 1.0 sec. PRT is reasonable as a minimum value for calculating the yellow change interval. However, the document acknowledges the significant amount of documentation regarding age-related increases in PRT. When engineering judgment determines a special need to take aging drivers' reduced capacity into consideration, the report suggests use of a 1.5 sec. PRT can be justified.

Caird, Chisholm, Edwards, and Creaser[29] examined older-driver PRT in a controlled, experimental study using a driving simulator. Their findings also suggested age and PRT are not correlated, except when the time to the intersection is long. The experiment involved 77 drivers approaching a test intersection at approximately 70 km/h, or 43.5 mph. The yellow indication was displayed for six values of time to the stop line, ranging from 1.73 to 3.58 sec. The reported mean PRT was 0.96 sec., with a range of 0.5 to 2.2 sec., and the 85th percentile PRT was 1.22 sec. Based on the results, the researchers concluded a 1 sec. PRT appears to be sufficient for all drivers.

Gates, Noyce, Laracuente, and Nordheim[30] conducted an observational study of 898 drivers at six intersections in Madison, WI, USA. Results indicated that approach speed, distance to the intersection, deceleration rate, and vehicle type were related to PRT. Gates et al. observed drivers at intersections with approach speeds ranging from 25 to 50 mph (40 to 80.5 km/h). The reported median PRT was 1.0 sec., and the 85th percentile PRT was 1.6 sec.

El-Shawarby, Amer, and Rakha[31] examined driver PRT in a controlled, experimental, study of 60 drivers. Study results suggested no significant relationship between PRT and age or gender, although PRT and the time to the stop line had a direct relationship. The experiment involved drivers approaching the test intersection at approximately 45 mph (72.4 km/h). The yellow indication was displayed for five values of the distance to the intersection, ranging from 32 m to 111 m. The time to the stop line ranged from 1.34 to 6.19 sec. The reported mean PRT was 0.73 sec., with a range of 0.14 to 2.4 sec.

The FHWA memorandum issued in 2008, and revised on July 1, 2009, on determining yellow change intervals suggests using a PRT of 1.0 sec. or greater.[32]

Research was performed as part of *NCHRP Report 731*[4] and further documented by Gates et al.[33] at 83 sites around the U.S., based on an initial data set of 7,482 vehicle reports. Reaction time was measured for more than 2,400 drivers in the decision zone that were first to stop after the yellow onset.

The research found

- PRT's (measured as brake-response time in the report) observed values were in agreement with, though slightly shorter than, previous studies;
- PRT decreased as approach speed increased (i.e., faster drivers reacted more quickly);
- PRT increased as travel time to the intersection at the start of yellow increased (i.e., drivers reacted more slowly when farther from the intersection);
- PRT Increased as deceleration rate increased (i.e., drivers decelerating more rapidly used longer PRT times); and
- PRT decreased for steep downgrades.

The mean PRT was 1.0 sec. with a standard deviation of 0.37 sec. and an 85th percentile PRT of 1.33 sec. In Gates et al.[33] documentation of the data analysis for the NCHRP report, he notes that

> *"the PRT and deceleration rate should be jointly considered as motorists do not select these variables independently of each other. Slow-reacting drivers tend to compensate with greater deceleration rates, and quick-reacting drivers tend to decelerate more comfortably. In either case, the decision to stop and subsequent braking occur over approximately the same overall time and distance."*

Gates observes that Parsonson[34] published a discussion of Wortman and Matthias'[35] paper on driver behavior regarding this specific principle. He concludes that the selection of PRT and deceleration rates should be based on centralized values (e.g., mean or median) for each parameter rather than more extreme values (e.g., 85th percentile or 15th percentile) and thus recommends a PRT of 1.0 sec.

Current Practice

Survey respondents were asked, if they used the kinematic equation, what value was used for perception-reaction time. One hundred respondents answered this question (Table 2.3). The overwhelming majority used 1.0 second.

Table 2.3: Perception-Reaction Times Used in Practice

Perception-Reaction Time	no. of responses	%
1.0 sec.	81	81
1.5 sec.	8	8
1.8 sec.	4	4
2.0 sec.	2	2
2.5 sec.	4	4
3.0 sec.	1	1
Total	**100**	**100**

Members also commented that changes in the PRT value affect the yellow change interval and that the PRT is the only human factor considered by the kinematic equation. Comments concerning PRT surrounded recent studies of driver PRT values in reaction to the onset of the yellow signal.

Recommendation

Recent observational studies on PRT support the value of 1.0 sec. as representative of the general driving population. The PRT affects only the yellow change interval, which provides time for the driver to perceive and react to the onset of the yellow indication and to either proceed through the intersection or begin stopping. The red clearance interval theoretically provides time for drivers to clear the intersection once they have entered prior to termination of the yellow change interval, which is not affected by PRT.

Based on the available research, a minimum PRT of 1.0 sec. is sufficient for most users given its strong correlation to the deceleration rate. This perception-reaction time is also the most widely used based on the survey findings. However, if local conditions, driving population age, or a supporting engineering study suggest a value higher than 1.0 sec. is appropriate, engineering judgment may be used to modify this value upward. Additionally, please refer to the discussion in Section 2.14 for PRT values associated with left-turn movements.

2.6 Speed

Literature

A History of the Yellow and All-Red Intervals for Traffic Signals[3] provides a comprehensive history of the approach speed variable used in change interval calculations. According to the report, the 85th percentile speed is commonly used today, although the recommended value has changed over the last 60 years.

ITE's *Determining Vehicle Change Intervals: A Proposed Recommended Practice*[1] states the 85th percentile speed is most representative of the approach speed, but additionally notes the posted speed limit may be preferred to avoid extensive field work. The report also suggests different approach speeds may be appropriate for calculating the yellow change and red clearance intervals. Two variations are suggested: The first method recommends calculating the total change period using the 85th percentile speed and the 15th percentile speed, and applying the greater of the two values based on the work of Parsonson and Santiago;[36] the second method modifies the first, recommending that if the value calculated from the 15th percentile speed is greater, the red clearance interval calculated from the 85th percentile speed should be increased by the difference based on the work of Butler.[37] A single recommendation on approach speed is not provided; rather, the ITE's report advocates the use of engineering judgment in determining an appropriate approach speed.

NCHRP Report 504: Design Speed, Operating Speed, and Posted Speed Practices[38] reported a strong relationship between operating speed (i.e., the 85th percentile speed), and the posted speed limit. The study assessed speed data from 79 tangent sections of various roadway classifications with varying speed limits in seven cities across six states in the United States. For all road classifications, the relationship between the 85th percentile speed and the posted speed limit was modeled by Equation 32.

$$EV85 = 7.675 + 0.98 \times PSL \qquad (32)$$

Where:

$EV85$ = 85th percentile speed (mph); and

PSL = posted speed limit (mph).

The regression indicates the 85th percentile speed is approximately 7 mph greater than the posted speed limit. This relationship was reflected in about half of the study sites which had a posted speed limit between 4 to 8 mph (6.4 to 12.9 km/h) below the 85th percentile speed. Researchers observed a greater percentage of vehicles on rural roads (37 to 64 percent) traveled at or below the posted speed limit compared to vehicles on suburban or urban roads (23 to 52 percent). The report also provides individual regression models for each functional class.

Tignor and Warren[39] presented the results of a study showing that speed limits on average were posted 8 to12 mph (12.9 to 19.3 km/h) below the 85th percentile speed, with the largest differences found on lower-speed facilities. The average difference over all 48 sites in the study was 9.5 mph (15.3 km/h). The FHWA memorandum[32] on determining yellow change intervals provided the following guidance statement based on the Tignor and Warren study:

> *"The minimum length of yellow should be determined using the kinematics formula in the 1984 ITE proposed practice assuming an average deceleration of 10 ft./sec./sec. or less, a reaction time of 1 sec. or more, and an 85th percentile approach speed. If the approach speed is not known, the posted speed limit plus 10 mph may be used."*

Research performed as part of *NCHRP Report 731*[4] examined approach speed for 83 sites and a data set of 3,632 through-movement vehicles in the study. The researchers concluded speed limit provides a good estimate of the mean approach speed of free-flowing vehicles arriving at a traffic signal. Based on the data, the speed limit on its own generally did not provide an accurate estimate of the 85th percentile approach speed. The 85th percentile approach speed was accurately predicted by adding 7 mph (11 km/h) to the speed limit at all speed limits except 25 mph (40 km/h) where adding 10 mph (16.1 km/h) was more consistent with the 85th percentile approach speed. The study concluded that in lieu of field-measured speed data the approach speed limit plus 7 mph (11 km/h) can be used as a rule of thumb for the purposes of timing traffic signal change intervals for through-moving vehicles.

This study states that the speed for the red clearance interval calculation for through vehicles should be the same as that for the yellow change interval, as through vehicles entering an intersection after the yellow has been displayed do not reduce their speed. The study did not measure speed data for the completion of the movement along a turning path through the intersection. Instead, the researchers calculated the 85th percentile value of the AASHTO horizontal curve design speed equation at 18.5 mph (29.8 km/h). Citing the conservative nature of the values calculated with the AASHTO equation due to the design side-friction factor used, the study recommended 20 mph (32.2 km/h) be used as the estimate for the 85th percentile for timing the red clearance interval regardless of approach speed limit.

Current Practice

As part of the survey, agencies were asked what value they used for the approach speed, if speed is a factor in the calculation of the change interval. The majority of respondents, 133 of 240 (55 percent), indicated they used the posted speed limit. Responses are shown in Table 2.4.

Table 2.4: Approach Speed Used in Practice

Speed	no. of responses	%
Posted speed limit	133	55
85th percentile approach speed	59	25
Design speed	6	2
Other	42	18
Total	**240**	**100**

Table 2.5: Speed Measures Used in Practice for the Calculation of Change Interval Duration

Samples of Speed Measures	
85th percentile where available	Posted speed for new, operating speed for existing
85th percentile for yellow, posted speed for red	Posted speed limit est. based on the 85th percentile speed
85th percentile or posted speed limit	Posted speed limit plus 5 mph (8 km/h)
85th percentile if known; if not, then posted speed	Posted speed on through movement
90th percentile for yellow	Posted speed plus 5 mph (8 km/h)
All of the above, depending on availability at design	Posted speed unless other information is given
British Columbia Ministry of Transportation—*Electrical and Traffic Engineering Manual*	Posted speed, unless85th percentile speed is known
85th percentile and posted speed limit	Posted unless engineering judgment dictates other
Either Posted or *prima facie* or 85th percentile	Posted unless speed evaluation is available
Engineering judgment	Posted, unless known 85th percentile higher (higher # used)
Estimated 15th percentile and 85th percentile speeds, whichever yields longer time	Recommended as 85th percentile or 15th percentile speed
For yellow change and red clearance intervals, we "follow" ITE recs (engineering judgment used)	See policy form
Greater of speed limit or 85th percentile (if known)	Speed limit with consideration to 85th percentile speed
High range of comfortable speed after trial runs	Through = posted speed
Info sent in separate email	Usually posted speed limit or 85th percentile when handy
Mostly 85th percentile, but use posted speed in special situations	Varies by location (85th percentile or Posted speed limit)
Posted + 5 mph (8 km/h) for amber, posted for red clearance interval	We estimate speed from speed limit and familiarity
Posted + 5 mph (8 km/h)	We use posted, but considering using 85th percentile.
Posted or *prima facie* speed	We use the greater of 85th percentile or posted
Posted speed for throughs or rights and lower speed for lefts	Yellow interval, posted speed or *prima facie* speed
Posted speed and observation	Yellow V = Posted + 5 mph (8 km/h); Red Clearance Interval V = Posted

Respondents who answered "other" to the preceding question regarding speed measures were asked to provide information regarding other methods used. Information regarding these other methods is summarized in Table 2.5.

Respondents who used a different speed measure to calculate the red clearance interval were asked to specify what speed measure was used. This was a multiple-choice question with choices and responses summarized in Table 2.6. About half the respondents used posted speed limits, compared with 18 percent that used 85th percentile approach speeds.

Table 2.6: Speed Measures Used in Calculation of Red Clearance Interval (if different than measures used in calculation of yellow)

Speed Measure	no. of responses	%
Posted speed limit	79	52
85th percentile approach speed	27	18
Design speed	2	1
Other	43	29
Total	**151**	**100**

Respondents who answered "other" to the preceding question regarding speed measures were asked to provide information regarding other methods used. Information regarding other methods is summarized in Table 2.7.

Most of the comments received on this topic surround whether to use the posted speed limit or some measure of speed

Table 2.7: Speed Measures Used in the Practice for Calculation of Red Clearance Interval (if different than measures used in calculation of yellow)

Samples of Speed Measures	
(In those rare cases), trial runs at "low" speed	N/A to us
0.5 sec. for left turn; 1 to 2 sec. for through phases	Not used
0.5 sec. left turns; 1 sec. through lanes	Posted - 10 mph (16 km/h)
0.5 sec. for turning movement and 1.0 sec. for through movement	Posted speed for new, operating speed for existing
10th percentile for red clearance	Posted speed for through, 15 mph (24 km/h) for left turns
50th percentile approach speed	Posted unless engineering judgment dictates other
50th percentile speed	Posted unless speed evaluation is available
85th percentile but time generally not to exceed 2 sec.	Recommended as 85th or 15th percentile speed
85th plus width of street	Red = 1.0 to 2.0 sec.
85th percentile if known; if not, then posted speed	Same
British Columbia Ministry of Transportation—*Electrical and Traffic Engineering Manual*	Same as above but -1 sec. for left turns only
85th percentile approach speed and speed limit	Same speed
Default value is 1.0 sec. and 2.0 sec. if needed	See policy form
Engineering judgment	Time from limit line to last point of collision at 10th percentile speed
Engineering judgment on various factors	Typically, we use a 2.0 sec. all-red interval
Field observations	Uniform 1.0 sec. unless accident problems persist
Follows the same as yellow change	Uniform setting of 2-sec.
For red clearance interval, we "follow" ITE recs (engineering judgment used)	Use same
For left turns, turn execution speed (see below)	Usually posted speed limit or 85th percentile when handy
Greater of speed limit or 85th percentile (if known)	Varies: Some use fixed values / incorporated speed
	Your survey doesn't provide enough room to answer.
Mostly 85th percentile, but use posted speed in special situations	Posted speed limit

collected in the field such as the 85th percentile. A broad range of comments were received, suggesting various speed measures or a hybrid approach using different speed measures for unique conditions. Other comments addressed the difference in speed-setting policies, such as in urban areas where speeds may be set by ordinance rather than by 85th percentile speeds. Several comments were concerned with an agency's ability to collect speed in the field due to limited resources and others were concerned about the variability of the speeds in the field if collected.

Jurisdictions that set speed via ordinances through an elected governmental body may have posted speed limits that differ from actual approach speeds. In these locations, it may be preferable to conduct a speed study.

Recommendation

The preferred method for representing approach speed is to use the 85th percentile approach speed for the yellow change interval and red clearance interval calculation. Spot speed data to support engineering studies to determine an 85th percentile approach speed can be collected by various methods, including RADAR, LIDAR, paired loop detectors, microwave detectors, and other tools. Speed is represented in the numerator for the yellow change interval calculation and the denominator for the red clearance interval calculation so the 85th percentile speed provides a consistent value.

If the 85th percentile speed is unavailable and a speed study is not conducted, the 85th percentile approach speed for through movements may be estimated by the following equation for calculating the yellow change interval:

$$V_{85}\text{ (through)} = SL + 7 \text{ (}\textit{U.S. units}\text{)} \quad (33)$$

Where:

V_{85} = 85th percentile speed (mph); and

SL = posted speed limit (mph).

V_{85} (through) = $SL + 11$ (*Metric units*) (34)

Where:
V_{85} = 85th percentile speed (km/h); and
SL = posted speed limit (km/h).

The relationship between 85th percentile speed and the posted speed limit is based on the results of field observational studies at 83 intersections documented in *NCHRP Report 731*[4] and supported by other research studies. This will provide sufficient yellow time for vehicles traveling at the assumed 85th percentile speed. Using the relationship between the 85th percentile speed and the posted speed limit allows an engineer to calculate the yellow change interval for a significant number of signalized intersections when approach speeds from field measurements are not available for every intersection approach. The policy decision by an agency to implement this practice should be made in the context of the roadway's characteristics and classification, applicable speed limit laws, agency speed limit engineering process, available resources, and the application of engineering judgment.

Speed is represented in the numerator for the yellow change interval calculation and the denominator for the red clearance interval calculation so the 85th percentile speed provides a consistent value.

The speed values used for the red clearance interval of through vehicles are based on the 85th percentile speed approach speed. This allows vehicles traveling through the intersection at the 85th percentile speed to traverse the intersection during the red clearance if they entered the intersection on yellow. If more speed studies demonstrate a different speed through the intersection, the design engineer should use judgment to apply the new primary data to the calculation.

2.7 Deceleration

Literature

The literature review found numerous early studies supported a 10 ft./sec./sec. (3 m/sec./sec.) deceleration rate. Most recent field studies of deceleration found observed deceleration rates vary greatly and are related to other variables unique to the roadway environment such as approach speed, roadway geometry, pavement surface friction, and distance or time to the intersection. Differences in deceleration rate by age and gender were also found.

As noted in *A History of the Yellow and All-Red Intervals for Traffic Signals*,[3] the deceleration rate of an approaching vehicle has the greatest effect on the variance of the calculated change interval.[40] The most recent guidance in national publications suggests applying a value of 10 ft./sec./sec. (3m/sec./sec.).

The 1941 and 1950 editions of the ITE *Traffic Engineering Handbook* incorporated deceleration rates as constants within the equation or as a variable of the stopping distance. Gazis et al.[7] concluded a 10 ft./sec./sec. (3 m/sec./sec.) deceleration rate was appropriate based on an observational study of 87 drivers. The 1965 edition of the *Traffic Engineering Handbook*,[10] however, suggested 15 ft./sec./sec. (4.6 m/sec./sec.) as a reasonable deceleration rate. *A History of the Yellow and All-Red Intervals for Traffic Signals*[3] cites Parsonson and Santiago[36] for suggesting that the source of 15 ft./sec./sec. (4.6 m/sec./sec.) deceleration rate was an earlier emergency stopping distance calculation method. The authors assert the value was then erroneously applied to the yellow change interval. Therefore, the *Manual of Traffic Signal Design*[41] was modified and suggested applying a 10 ft./sec./sec. (3 m/sec./sec.) deceleration rate, which was supported by Gazis et al.[7] All subsequent editions of the *Traffic Engineering Handbook* have retained this 10 ft./sec./sec. (3 m/sec./sec.).

The 1983 observational study by Wortman and Matthias[35] reported mean deceleration rates ranging from 7.0 to 13.8 ft./sec./sec. (2.1 to 4.2 m/sec./sec.) for approach speeds of 30 to 50 mph (48.3 to 80.5 km/h). The 85th percentile deceleration rates were 11.5 to 18.2 ft./sec./sec. (3.5 to 5.5 m/sec./sec.).

Findings from an observational study by Chang, Messer, and Santiago[25] agree with the Wortman and Matthias study. For approach speeds of 30 to 55 mph (48.3 to 88.5 km/h), the mean deceleration rate was 9.5 ft./sec./sec. (2.9 m/sec./sec.).

Referenced in the *Handbook for Designing Roadways for the Aging Population*,[42] Knoblauch et al.[28] observed higher deceleration rates in an experimental study. Mean deceleration rates from the study ranged from 10.7 to 15.2 ft./sec./sec. (3.3 to 4.6 m/sec./sec.).

The 2007 experimental study by Caird, Chisholm, Edwards, and Creaser[29] also examined deceleration rates for 77 drivers approaching an intersection at 70 km/h (43.5 mph). Findings supported a significant relationship between deceleration rate and time to stop line and age. Deceleration rates decreased as drivers were farther from the stop line. For a range of controlled time to stop line values, mean deceleration rates ranged from 8.2 to 18.0 ft./sec./sec. (2.5 to 5.5 m/sec./sec.). Results indicated deceleration rates were slower for older drivers. The mean deceleration rate for 18 to 35 year old drivers was 14.4 ft./sec./sec. (4.4 m/sec./sec.), compared to 12.5 ft./sec./sec. (3.8 m/sec./sec.) for 55 to 64 year old drivers and 12.3 ft./sec./sec. (3.7 m/sec./sec.) for 65 year old and older drivers.

El-Shwarby, Rakha, Inman, and Davis[43] analyzed driver deceleration behavior at the onset of the yellow indication in a controlled, experimental study of 60 drivers on a 45 mph (72.4 km/h) approach. Results suggested a relationship between deceleration rate and time to the stop line, driver age, and driver gender. Similar to Caird et al.'s results, drivers had slower deceleration rates when the time to the stop line was greater. However, unlike results of the Caird et al. study, both younger (age 40 and younger) and older (age 60 and older) drivers had greater deceleration rates compared to middle-aged (age 40 to 59) drivers. The mean deceleration rate of 10.7 ft./sec./sec. (3.3 m/sec./sec.) was similar to the ITE-suggested value of 10 ft./sec./sec. (3.0 m/sec./sec.), with a range of 5.0 to 24.5 ft./sec./sec. (1.5 to 7.5 m/sec./sec.).

Results from an observational study by Gates, Noyce, Laracuente, and Nordheim[30] indicated a strong relationship between deceleration rate and approach speed. For approach speeds of less than 40 mph (64.4 km/h), the 50th percentile deceleration rate was 10.9 ft./sec./sec. (3.3 m/sec./sec.), while the 85th percentile deceleration rate was 13.6 ft./sec./sec. (4.2 m/sec./sec.). When approach speeds increased to 40 mph (64.4 km/h) or greater, the 50th percentile deceleration rate decreased to 8.3 ft./sec./sec. (2.8 m/sec./sec.), while the 85th percentile deceleration rate decreased to 11.6 ft./sec./sec. (3.5 m/sec./sec.).

The fifth edition of AASHTO's *A Policy on Geometric Design of Highways and Streets*, also known as the *Green Book*, suggests an 11.2 ft./sec./sec. (3.4 m/sec./sec.) comfortable deceleration rate for calculating the stopping sight distance.[44] No guidance is given on applying this value for calculating change intervals.

NCHRP Report 731[4] examined this parameter as well and was further documented by Gates et al.[33] at 83 sites around the United States based on an initial data set of 7,482 vehicle reports. Deceleration was measured for more than 2,400 drivers in the decision zone that were first to stop after yellow onset. The results were similar to earlier research, with a mean deceleration rate of 10.08 ft./sec./sec. (3.07 m/sec./sec.) and an 85th percentile value of 12.89 ft./sec./sec. (3.93 m/sec./sec.). The research also found deceleration:

- increased as approach speed increased (i.e., faster drivers used greater deceleration);
- decreased as travel times to the intersection at the start of yellow increase (i.e., drivers used lower deceleration when farther from the intersection); and
- increased as PRT increased (i.e., slower-reacting drivers used greater deceleration rates). PRT and deceleration rate were found to be directly correlated with each; time-of-day factors had limited impact on them.

The study authors recommend the use of the mean value of the deceleration rate and proposed the use of a 10 ft./sec./sec. (3 m/sec./sec.) rate.

Current Practice

The survey respondents were asked about deceleration rate for the kinematic equation. The overwhelming majority reported using 10 ft./sec./sec. (3 m/sec./sec.) or a similar value (Table 2.8).

Table 2.8: Values Used for Deceleration Rate

Deceleration Rate	no. of responses	%
9.8 ft./sec./sec. (3 m/sec./sec.)	5	5
10.0 ft./sec./sec. (3 m/sec./sec.)	84	78
10.02 ft./sec./sec. (3.1 m/sec./sec.)	4	4
11.2 ft./sec./sec. (3.4 m/sec./sec.)	11	10
20.0 ft./sec./sec. (6.1 m/sec./sec.)	1	1
Other	2	2
Total	**107**	**100**

Primary comments received were related to the concern that deceleration rate would need to be measured in the field.

Recommendation

Guidance on applying the deceleration rate typically provides average values for deceleration rate rather than measurement of the rate in the field. Based on the available research, a uniform deceleration rate of 10 ft./sec./sec. (3 m/sec./sec.) is appropriate for most users. The kinematic equation assumes uniform deceleration though this is an oversimplification. However, a uniform deceleration rate of 10 ft./sec./sec. (3 m/sec./sec.) is a mean rate correlated to perception-reaction time. Further, researchers found that if the required deceleration was greater than 12 ft./sec./sec. (3.7 m/sec./sec.) most drivers would go, and if the deceleration was less than 8 ft./sec./sec. (2.4 m/sec./sec.) most drivers would stop. The values recommended are based on a 10 ft./sec./sec. (3 m/sec./sec.) deceleration rate.

2.8 Intersection Width

Literature

A History of the Yellow and All-Red Intervals for Traffic Signals[3] provides a summary of past guidance on the width of the intersection parameter. The report notes that minimal guidance has been provided in past editions of the ITE *Traffic Engineering Handbook*, and suggests guidance could be strengthened in the future.

Definitions and guidance from U.S. publications on intersection width vary. Measurement of intersection width may begin at the intersection entry point defined as the stop line, crosswalk threshold, or near-side conflicting curb line. For through movements, measurement may extend to far-side conflicting crosswalk line. The ITE *Traffic Engineering Handbook,* 6th Edition[8] recommends intersection width should be measured along vehicle path from stop line to far-side no-conflict point.

The ITE *Traffic Engineering Handbook*, 4th Edition[9] provides three equations for calculating the red clearance interval based on the presence of pedestrians (Equations 9 and 10).

The 2009 MUTCD as revised[5] defines the intersection width for an intersection controlled by a traffic signal as

1. *If a stop line, yield line, or crosswalk has not been designated on the roadway (within the median) between the separate intersections, the two intersections and the roadway (median) between them shall be considered as one intersection;*
2. *Where a stop line, yield line, or crosswalk is designated on the roadway on the intersection approach, the area within the crosswalk and/or beyond the designated stop line or yield line shall be part of the intersection; and*
3. *Where a crosswalk is designated on a roadway on the departure from the intersection, the intersection shall include the area extending to the far side of such crosswalk.*

The UVC[6] defines an intersection as

> *"The area embraced within the prolongation or connection of the lateral curb lines, or if none, then the lateral boundary lines of the roadways of two highways which join one another at, or approximately at, right angles, or the area within which vehicles travelling upon different highways joining at any other angle may come in conflict."*

NCHRP Report 731[4] noted the *Traffic Engineering Handbook,* 6th Edition definition (see above) and five other intersection-width definitions from national resource publications, from shortest to longest along the vehicle path:

- Curb-line extension to curb-line extension;
- Near-side stop line to the middle of the first conflicting traffic lane;
- Near-side stop line to the far edge of the last conflicting traffic lane;
- Near-side stop line to the far-side curb-line extension; and
- Near-side stop line to the far side of the far-side crosswalk, if one exists.

The report discusses the implications of the various options on vehicle clearance, pedestrians, start-up delay and other factors, including consideration for blind or visually impaired pedestrians. The report proposed the intersection width "be measured from the upstream edge of the approaching movement stop line to the far side of the intersection as defined by the extension of the curb line or the outside edge of the farthest travel lane." For left-turning vehicles, the report suggests using the approaching movement turning path distance between these same points.

Current Practice

Survey respondents were asked whether they measured intersection width in the field. Nearly half of respondents, 132 of 267 (49 percent), reported doing so. A minority, 18 of 267 (7 percent), measure the crosswalk distance or width. The primary concern expressed in the survey responses and by the practitioners working for public agencies was the availability of resources to collect primary data in the field.

Recommendation

Field measurements with an apparatus of choice provide the most accurate road width distance. However, as-built design plans, aerial photography, GPS, and surveys enable practitioners to gather measurements of intersection width with minimal resources.

Intersection width has a large effect on the duration of the red clearance interval (or the total change period in the case of *restrictive* laws, or jurisdictions that do not use a red clearance interval). The preferred method is to measure the total distance from the stop bar to the curb-line extension or outside edge of the farthest conflicting traffic lane, along the vehicle's travel path. Figure 2.2 illustrates this distance.

2.9 Vehicle Length

Literature

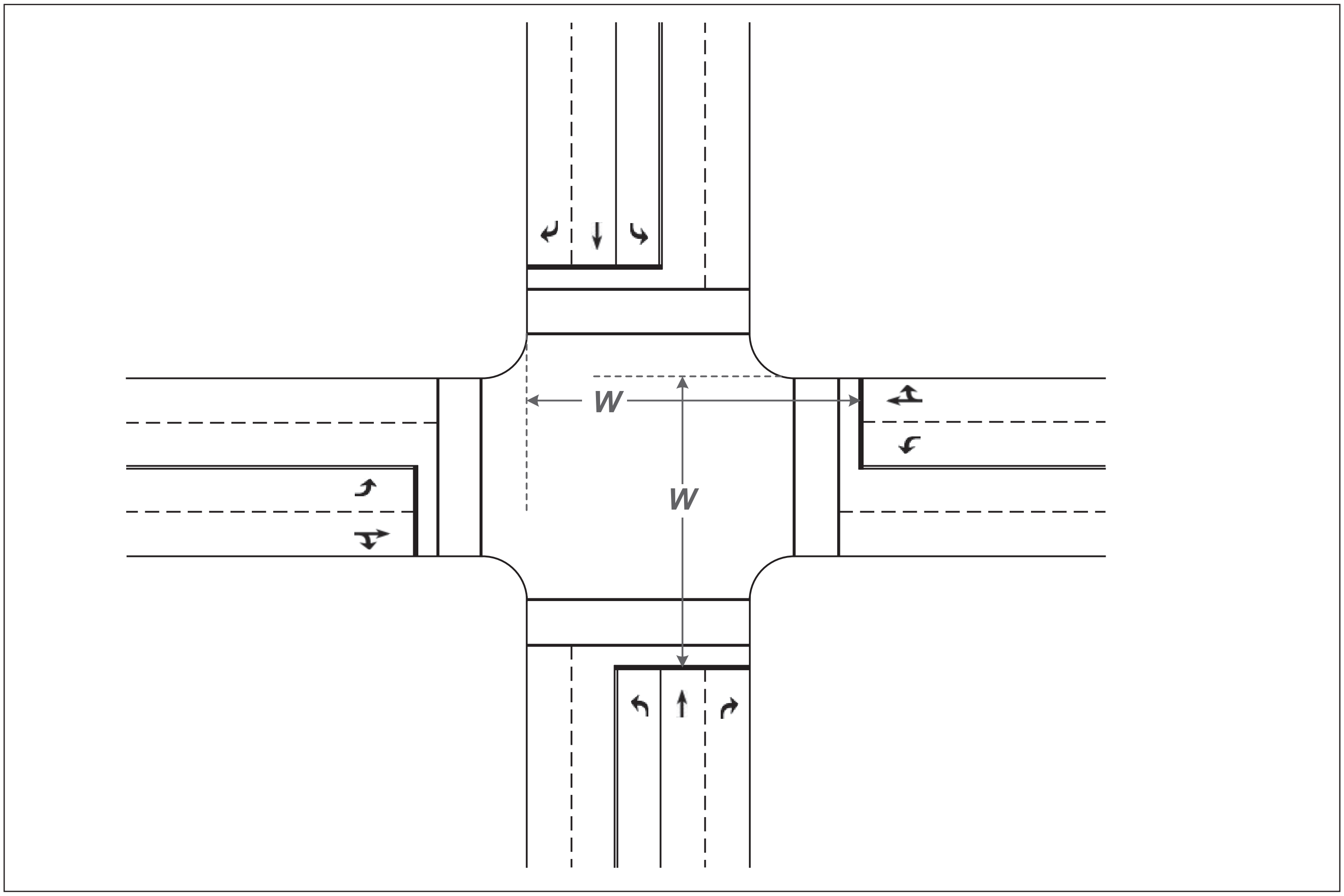
The vehicle length variable takes into account the length of a large majority of four-wheel vehicles that must clear the intersection or conflict point. In 1977, Williams[17] suggested a 17 ft. (5.2 m) vehicle length for use in his combined kinematic model and stopping probability method.

The 1965 edition of the *Traffic Engineering Handbook*[24] suggested a 20 ft. (6.1 m) vehicle length. Subsequent guidance by ITE retains the use of this value, with the exception of the first edition of the *Traffic Control Devices Handbook*[45], which suggests 15 ft. (5.2 m). The second edition of the *Traffic Control Devices Handbook*[12] uses 20 ft. (6.1 m) for vehicle length.

The *Green Book* provides groupings of selected vehicles ("design vehicles") to establish highway design controls.[44] The *Green Book* length of passenger car design vehicle is 19 ft. (5.8 m). The length of a WB-50 design truck for intersection design is 55 ft. (16.8 m). The length of a WB-65 or WB-67 minimum size design truck for intersections on state highways, industrialized streets, or streets that provide local access for trucks is 73.5 ft. (22.4 m).

NCHRP Report 731[4] noted the *Green Book* values and notes that considering longer vehicles in the calculation would increase the duration of the red clearance interval. The report states "…conflicting vehicle traffic is obligated to yield the right-of-way to other vehicles legally in the intersection," thus making the statutory requirement the controlling factor. The authors proposed using the value of 20 ft. (6.1 m) for vehicle length.

Figure 2.2: Diagram of Intersection Width for Through Movements



Current Practice

Respondents were asked what value their agency uses for vehicle length if they apply the kinematic equation method. A majority of respondents, 66 of 107 (62 percent), reported using a 20 ft. vehicle length (Table 2.9).

Table 2.9: Values Used for Vehicle Length

Vehicle Length	no. of responses	%
0 ft. (0 m)	5	5
18 ft. (5.4 m)	1	1
19.7 ft. (6.0 m)	8	7
20 ft. (6.1 m)	66	62
22 ft. (6.7 m)	2	2
25 ft. (7.6 m)	10	9
45 ft. (13.7 m)	1	1
other	3	3
not used	11	10
Total	**107**	**100**

Recommendation

A vehicle length of 20 ft. (6.1 m) is sufficient for most users. Longer vehicle length may be considered based on a supporting vehicle classification study and application of engineering judgment.

2.10 Grade

Literature

A History of the Yellow and All-Red Intervals for Traffic Signals[3] references the 1982 edition of the *Manual of Traffic Signal Design*[41] for the first inclusion of grade in calculating changing intervals. The report suggests consideration of grade may have been the result of work by Parsonson and Santiago.[36] Subsequent ITE publications have included the approach grade variable in the kinematic equation calculation method. Grade is included in the denominator of the second term in the kinematic equation.

The FHWA *Traffic Signal Timing Manual*[11] suggests adding 0.1 sec. to the calculated yellow change interval for every 1.0 percent downgrade, and conversely, subtracting 0.1 sec. from the calculated yellow change interval for every 1.0 percent upgrade.

The field study conducted as part of the *NCHRP Report 731*[4] research showed grade had an impact on PRT and deceleration rates. The researchers identified upgrades and downgrades greater than 3 percent as resulting in deceleration rates different from those for level terrain. The report authors did not suggest grade modification factors for PRT and deceleration rates; however, they supported the continued use of grade in the kinematic equation. The authors further suggested the grade measurement be taken at the distance corresponding to the upper boundary of the indecision zone.

Current Practice

The survey asked respondents what data was collected in the field prior to timing change intervals. A minority of respondents, 30 of 267 (11 percent), reported measuring grade in the field.

All the comments received were regarding field data collection of the approach slope, including the resources which would be needed.

Recommendation

A standard way of collecting intersection approach grade does not exist. The preferred method is to field measure the approach grade for existing roads or use the design approach slope grade for proposed roads measured from the upper boundary of the indecision zone (the critical distance) and use the value in the kinematic equation (Equation 6). Alternatively, approach grade may be taken from an as-built roadway design plan or other document that specifies design criteria.

2.11 Minimum and Maximum Intervals

Literature

Section 4D.26 of the 2009 MUTCD[5] provides the following guidance on minimum and maximum yellow change and red clearance intervals:

> *"A yellow change interval should have a minimum duration of 3 seconds and a maximum duration of 6 seconds. The longer intervals should be reserved for use on approaches with higher speeds.*
>
> *Except when clearing a one-lane, two-way facility (see Section 4H.02) or when clearing an exceptionally wide intersection, a red clearance interval should have a duration not exceeding 6 seconds."*

The literature review conducted as part of the research for *NCHRP Report 731*[4] did not find any support for these suggested values. The study's authors did not suggest minimum or maximum values for the yellow change interval. The report did suggest a minimum value of 1.0 sec. for the red clearance interval even if the calculated value was less than 1.0 sec., to provide a safety factor before the release of any conflicting traffic. No maximum red interval was suggested in the report.

Current Practice

Respondents were asked what, if any, were their minimum and maximum values for yellow change intervals, red clearance intervals, and total change period. Responses are summarized in Table 2.10.

Minimum yellow timing values ranged from 1.5 to 4.0 sec. with 71 percent of respondents reporting minimum yellow timing values of 3.0 sec. (Values less than the MUTCD-required 3.0 sec. are probably referring to ramp meter timings or dummy phases at signalized intersections.) Maximum numeric yellow timing values ranged more broadly, from 3.0 to 7.0 sec., plus seven agencies reporting no maximum. The largest single response (38 percent) was 5.0 sec., with 77 percent of respondents reporting agency maximum yellow timing values ≥ 5.0 sec.

Table 2.10: Number of Agencies Reporting Minimum and Maximum Change Period Timing Values (Number of Responses)

Sec.	Yellow Change		Red Clearance		Total Change Period	
	Min	Max	Min	Max	Min	Max
0	—	—	34	—	—	—
0.1 to 0.9	—	—	40	—	—	—
1.0	—	—	123	15	—	—
1.1 to 1.9	1	—	12	5	—	—
2.0	2	—	9	75	—	—
2.1 to 2.9	1	—	1	9	—	—
3.0	163	1	—	10	19	—
3.1 to 3.9	31	2	—	5	18	—
4.0	29	23	—	10	65	—
4.1 to 4.9	—	12	—	2	15	—
5.0	—	75	—	5	25	3
5.1 to 5.9	—	18	—	—	2	2
6.0	—	59	—	13	3	19
6.1 to 6.9	—	—	—	—	—	10
7.0	—	2	—	—	7	33
7.1 to 7.9	—	—	—	—	—	11
8.0	—	—	—	—	—	21
> 8.0	—	—	—	—	—	16
None	—	7	2	1	10	25

Minimum red clearance time values ranged from 0 to 2.5 sec. The largest single response (57 percent) was 1.0 sec., with two-thirds of respondents reporting minimum red clearance time values ≥ 1.0 sec. A broad range of maximum red clearance time values was reported, ranging from 1.0 to 6.0 sec. The largest single response (51 percent) was 2.0 sec., with almost two-thirds of respondents reporting agency maximum red clearance time values ≤ 2.0 sec. One respondent (not included in Table 2.10) reported a maximum of 8.0 sec., but limited to single point urban interchanges.

Minimum values for total change period ranged from 3.0 to 7.0 sec., with 10 agencies reporting no minimum value. The largest single response (40 percent) was 4.0 sec., with 86 percent of respondents reporting minimum total change period from 3.0 and 5.0 sec. Maximum numeric values for total change period ranged from 5.0 to more than 8.0 sec., with 24 agencies reporting no maximum. Agencies with maximum values for total change period of 7.0 sec. or more, including those with no maximum, accounted for 76 percent of respondents.

Recommendation

The proposed recommended practice should use the ranges provided in the MUTCD[5] guidance for yellow change interval with the allowance for engineering judgment and/or study to address special road conditions. The minimum red clearance interval is proposed as 1.0 sec.

2.12 Rounding Calculated Intervals

Literature

NCHRP Report 731[4] notes modern traffic signal controllers can program settings to one-tenth of a second and the time for yellow change and red clearance intervals can be precisely calculated. The report recommends calculated values ending in 0.01 to 0.04 be rounded down to nearest 0.1 sec. and values ending in 0.05 to 0.09 be rounded up to nearest 0.1 second.

Further, it suggests a rounding approach for agencies that have a policy of rounding values to the nearest 0.5 sec.:

- Values ending in 0.0 to 0.1 should be rounded down to the nearest whole number;
- Values ending in 0.2, 0.3, and 0.4 should be rounded up to the half-second;
- Values ending in 0.6 should be rounded down to the half-second; and,
- Values ending in 0.7, 0.8, and 0.9 should be rounded up to the nearest whole number.

Current Practice

Comments addressed whether rounding results in significant differences and if so, when it is important. One commentator noted it is more important for the red clearance interval for left turns. Another commentator asked if rounding to the nearest tenth of a second is necessary, as drivers may not perceive such small differences. Other agencies round all hundredths up to the next tenth of a second.

Recommendation

The preferred rounding scheme is to round the final calculated interval up to the nearest 0.1 sec. Traffic signal controllers are typically capable of timings to the nearest 0.1 sec.

2.13 Use and Calculation of Red Clearance Interval

Literature

Crash-based research evaluations do not provide a clear indication of the safety effects of implementing red clearance intervals. The *NCHRP Report 731*[4] literature review notes previous studies have not definitively or consistently demonstrated long-term crash reductions associated with the use of red clearance intervals. The report also states that the speed for the red clearance interval calculation should be the same as that for the yellow change interval, as through vehicles entering an intersection after the yellow has been displayed do not reduce their speed. Most available studies have relatively weak experimental designs and other limitations. Of the available studies, results range from relatively large crash reductions, modest crash reductions, crash increases, to no effects. The strongest study on this topic, conducted by Souleyrette et al.[46] (which still has some methodological limitations), suggests modest short-term crash reductions, but no longer-term effects associated with using red clearance intervals. Absent more definitive research, the safety effects of installing red clearance intervals are inconclusive.

Intersection entry delay and start-up delay were examined as part of the research for *NCHRP Report 731*[4] to determine how much time delay occurs before the first vehicle enters an intersection after the onset of a green signal indication. The report found the start-up delay after start of green for stopped vehicles was 1.22 sec. and for stopped and rolling vehicles 1.10 sec. The total intersection entry delay after start of green for stopped vehicles was 4.38 sec. and for stopped and rolling vehicles, 4.10 sec. The report concluded a 1.0 sec. subtraction from the calculated red clearance interval was appropriate relative to greater than 4.0 sec. intersection entry delay. The report also cited studies that showed pedestrian entry delay values of 3 sec. in general and 1.93 sec. for younger pedestrians.

Fitch et al.[47] created a model that combines the two types of vehicles that may conflict at an intersection: the rolling start on green for a stopped vehicle when first seeing the green signal indication and the last vehicle clearing the intersection at the end of yellow, traveling at the low end of the speed distribution. The model effectively takes the worst case of the two conditions to create the red clearance interval, the slow vehicle clearing the intersection and the other vehicle starting quickly from the intersection. Results indicated the time to reach the intersection conflict point after the start of green from a complete stop at the stop bar was 2 sec. However, with a rolling start 12 ft. (3.7 m) from the stop bar, the conflict point is reached in 1.55 sec. (The formula used by Fitch et al. does not account for reaction time; times would be greater if included.) The article concluded this approach to timing the red clearance interval independently of the yellow change interval resulted in statistically significant reductions in collision, injury, and fatality rates at the study locations.

Current Practice

Survey respondents were asked if their agency had a formal policy on red clearance intervals. Of the 100 respondents to this question, the majority, 63 percent, did not. The range of comments received included that the red clearance intervals should always be included so drivers have enough time to clear the intersection. Comments also stated the red clearance interval does not need to be excessive because conflicting vehicles or pedestrians are required to yield to vehicles already in the intersection.

Recommendation

Crash-based studies are inconclusive about the safety effects of the red clearance interval. While the use of the red clearance interval may or may not have a positive effect on safety, agencies view it as desirable. Use of the red clearance interval is consistent with yellow change interval calculations under the *permissive* yellow laws and is recommended for use. The subchapter on minimum and maximum values recommends a minimum value of 1.0 sec. for the red clearance interval. A 1.0 sec. intersection entry delay factor should be subtracted from the calculated red clearance interval as long as the result is not less than 1.0 sec. Higher intersection entry delay values may be used based on engineering judgment or as supported by an engineering study.

2.14 Left-Turn Movements

Literature

Approach speeds for turning vehicles differ from through-movement vehicles. As part of the development of a proposed change interval calculation method for left-turning vehicles, Yu, Qiao, et al.[19,20,48,49] collected field data for 125 vehicles at 21 intersections in Texas. The data confirmed that left-turn approach speeds are lower than through-movement approach speeds. The mean approach speed for left-turning vehicles was reported to range from 29.37 mph (47.27 km/h) for 40 mph (65 km/h) speed limits to 36.24 mph (58.32 km/h) for 50 mph (80 k/h) speed limits, and the mean time required for making the left turn was 4.24 sec. They also created a mechanism to calculate the length of the turning path through an intersection based on clearance measurements and the angle of the intersection. Their method also suggested the use of the longest turning path with multi-lane left-turn approaches.

NCHRP Report 731[4] measured the speeds of approaching free-flow left-turning vehicles for speed limits between 40 mph (65 km/h) and 55 mph (90 km/h) at 19 locations for 570 vehicles. The research found these approach speeds were 4.94 mph (7.95 km/h) less than the posted speed limit and recommended the speed limit minus 5 mph (8 km/h) as the estimate of approach speed for left-turning vehicles for the purposes of timing the yellow change. Further, the report states, "In many cases, left-turning drivers are already braking at the onset of the yellow change interval, thereby greatly reducing or eliminating the PRT in response to the yellow indication."

Measuring intersection width for left-turn movements involves measuring curved vehicle path and identifying the vehicle's speed along that path. *NCHRP Report 731*[4] did not measure speed data for the completion of the movement along the turning path through the intersection. Instead, the researchers calculated the 85th percentile value of the AASHTO horizontal curve design speed equation at 18.5 mph (29.8 km/h) along the path from stop line to outside edge of the farthest travel lane. Citing the conservative values calculated with the AASHTO equation due to the design side-friction factor used, the study recommended 20 mph (32.2 km/h) be used as the estimate for the 85th percentile for timing the red clearance interval, regardless of approach speed limit.

The ITE *Traffic Engineering Handbook,* 6th Edition[8] recommends intersection width for exclusive turning movements be measured along vehicle path from stop line to far-side no-conflict point.

Additionally, *NCHRP Report 731*[4] provided guidance for the timing for left-turning vehicles that should take into account protected-only, permissive, and protected/permissive left-turn signal phasing in the development of yellow change and red clearance intervals, specifically:

- Calculate yellow change and red clearance intervals for protected only left-turn movements for each approach. The intervals can be different durations for opposing approaches.
- Calculate yellow change and red clearance intervals for permissive-only left-turn movements for opposing approaches, including the through movements. Use the longest calculated values for the different movements. The intervals must be the same duration for the left-turn and through movements on opposing approaches so termination is concurrent.
- Calculate yellow change and red clearance intervals for protected/permissive left-turn movements for the respective protected and

permissive portions of the phase and apply as described in the above two bullet points.

MUTCD[5] states

> *"When an actuated signal sequence includes a signal phase for permissive/protected (lagging) left-turn movements in both directions, the red clearance interval may be shown during those cycles when the lagging left-turn signal phase is skipped and may be omitted during those cycles when the lagging left-turn signal phase is shown."*

NCHRP Report 400: Determination of Stopping Sight Distances[50] and Koppa et al.[51] discuss the perception brake reaction time of an alerted driver from earlier research and from new primary data. The authors cite a surprise vs. anticipated perception/reaction/braking time (PBRT) ratio of 1.35, although in response to an auditory signal from the work of Johansson and Rumar. Applying this ratio to a 1.0 sec. PRT results in a value of 0.75 sec. for an alerted driver. Additionally, *NCHRP Report 400* also cites a comparison of study values (with some traffic signals as the object) for unsuspecting versus alerted drivers in both behind-the-wheel and simulator environments. The authors note that a study by Olson has surprise vs. alerted factor at 1.75. Applying this ratio to a 1.0 sec. PRT results in a value of 0.57 sec. for an alerted driver. That study collected new empirical data for PBRT for drivers approaching expected objects with mean results of 0.52 sec. for younger drivers, 0.66 sec. for older drivers, 0.59 sec. for male drivers and 0.63 sec. for female drivers. The weighted average across the study sample is 0.60 sec. Further, the Olson study data showed a mean PRT to an unexpected object (which included red signal onset) was stated as 1.1 sec.

Current Practice

Over a quarter of the survey respondents indicated they had a special policy for determining change intervals for left turns; 69 of 267 (26 percent) reported policies for left turns. Policies on timing change intervals for left-turn movements varied. Some policies suggested applying a lower approach speed, for example 15 mph (24.1 km/h), 20 mph (32.2 km/h), or 27 mph (43.5 km/h). Other policies used uniform values, for instance, a 3.0 sec. yellow change interval and 1.0 sec. red clearance interval for left turns. Policies also modified the definition of the intersection width for left turns, most suggesting measuring the vehicle turning path with possible inclusion of conflicting crosswalks. A few policies address left-turn phasing types and double left-turn lanes.

Only one of 267 respondents reported measuring the distance for left turns to clear the intersection. One respondent also reported the agency considers the number of left-turn lanes.

Many of the comments were related to concerns about avoiding a yellow trap for permissive left turns even though that is a signal phasing issue. Other comments addressed consistency between the left-turn interval and the intervals for the through vehicles, using the vehicle path instead of the intersection width in the calculation of a red clearance interval for the left turn, and the approach speed for the left turn in calculations.

Recommendation

The preferred method for representing approach speed is to use the 85th percentile approach speed for the yellow change interval for left turns. If the 85th percentile approach speed for the left-turn movement is unavailable and a speed study is not conducted, the 85th percentile approach speed for turning movements may be estimated as the speed limit minus 5 mph (8 km/h) by the following equation for calculating the yellow change interval:

$$V_{85}\,(\text{turn}) = SL - 5 \qquad (\textit{U.S. units}) \quad (35)$$

Where:
V_{85} = 85th percentile speed (mph); and
SL = posted speed limit (mph).

$$V_{85}\,(\text{turn}) = SL - 8 \qquad (\textit{Metric units}) \quad (36)$$

Where:
V_{85} = 85th percentile speed (km/h); and
SL = posted speed limit (km/h).

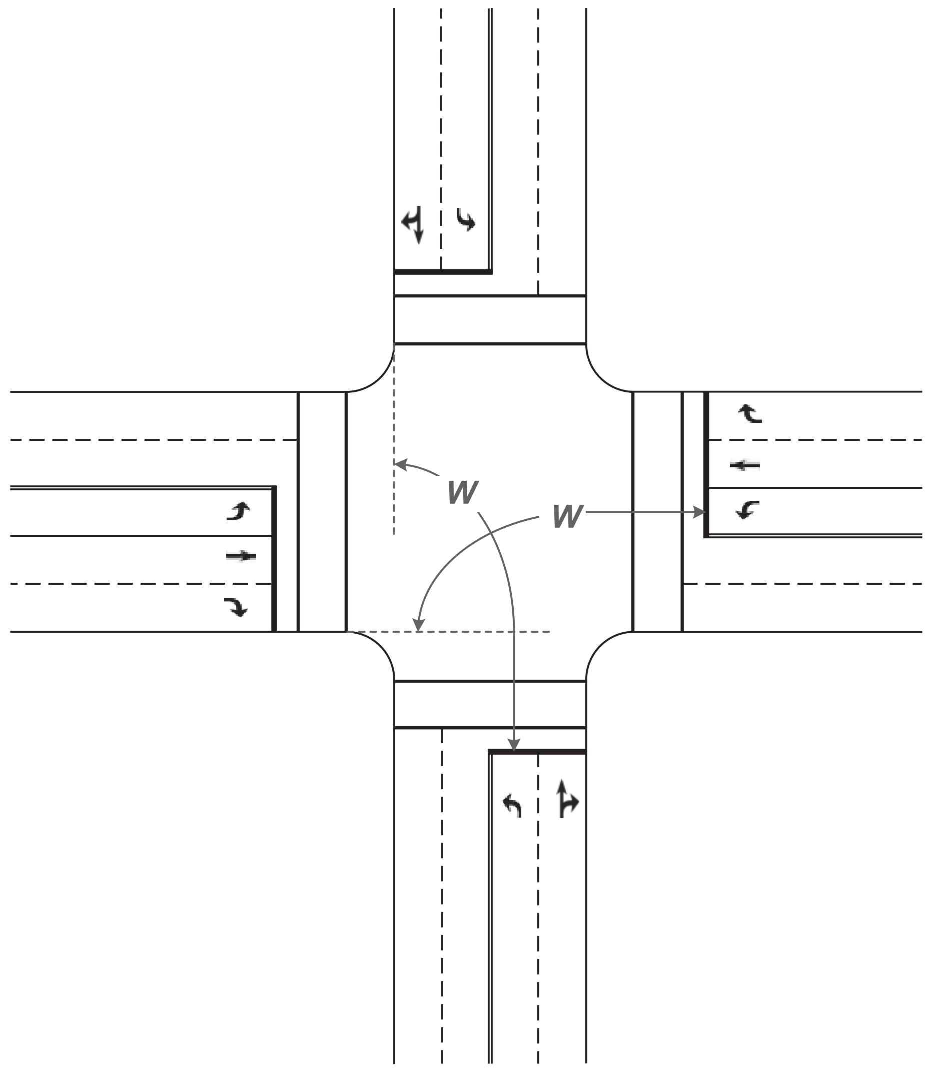
The speed of the turning vehicle should also take into account that the turning vehicle moves at a turning speed through the intersection lower than its approach speed. Therefore, speed for the red clearance interval of left-turning vehicles should be 20 mph (32.2 km/h) and distance should be measured along the centerline turning radius at the front axle to the departure leg curb- line extension. If speed studies demonstrate a different speed approaching or through the intersection, the design engineer should use judgment to apply the new primary data to the calculation. The relationships between 85th percentile speed and the posted speed limit are based on the results of field observational studies at 19 intersections documented in *NCHRP Report 731*[4] and supported by other research studies.

The value of PRT for the left-turning vehicles should be 0.6 sec., corresponding to an alerted driver expecting to make a left-turn movement.

The following notes the recommended approach to calculating the yellow change and red clearance intervals:

- Protected only left-turn movements: Calculate the yellow change and red clearance intervals for each approach and implement as calculated. The intervals can be of different duration for opposing approaches or adjacent through-movement phase.
- Permissive only left-turn movements: Calculate the yellow change and red clearance intervals for opposing approaches, including

Figure 2.3: Diagram of Left-Turn Movement Path



through movements, and use the longest of the calculated values (left, through, or combination). The intervals should be the same duration for the left-turn and through movements on opposing approaches to ensure that termination is concurrent.

- Protected/permissive left-turn movements: Calculate the yellow change and red clearance intervals and implement as described above for the respective protected and permissive portions of the phase. The implemented yellow change and red clearance intervals should be the longer of the calculated values for the left-turn and through- movement phases. The intervals should be the same duration for the left-turn and through-movement phases on opposing approaches to ensure that termination is concurrent.

While this approach may not take into account all possible left-turn signal phasing combinations, it provides a basis for the engineer to apply judgment in the development of yellow change and red clearance intervals for other phasing scenarios.

In the same manner as through movements, intersection width for left-turn movements is the total distance from the stop bar to the curb-line extension, or outside edge of the travel lane, of farthest conflicting movement along the vehicle's natural turning path. Figure 2.3 illustrates intersection width for left-turn movements. Where there are multiple lanes present, either on the approach or departure leg of the intersection, the longest distance should be used. Field measurements and verification of the turning path with an apparatus of choice provide the most accurate road width measure distance. However, as-built design plans, recent aerial photography, GPS, and surveys that reflect the current layout of the intersection enable practitioners to gather measurements of intersection width with minimal resources or field work safety concerns.

2.15 Other Road Users

Other road users include heavy vehicles, transit vehicles, older drivers, pedestrians, and bicyclists. Older drivers are addressed in previous sections on perception-reaction time and deceleration rate.

Literature

NCHRP Report 505[52] reported truck deceleration rates ranging from 5.44 to 11.52 ft./sec./sec. (1.66 to 3.51 m/sec./sec.) for various conditions. Findings also indicated trucks with antilock brakes can decelerate nearly as fast as passenger cars.

Intuitively, the PRT and deceleration rate of older drivers may differ from those of the overall driving population. However, the *Handbook for Designing Roadways for the Aging Population*[42] concluded change interval calculation methods did not need to be modified to accommodate older drivers.

The literature review included several studies that provided operating characteristics for bicyclists, including average speed, average deceleration, and 98th percentile speed. These operating characteristics may be important when bicyclists are part of the traffic stream. A study of 2,097 bicyclists conducted in Davis, CA, USA reported a lower average speed of 9.2 mph (14.8 km/h).[53] A smaller study of 28 bicyclists conducted in Mountain View, CA, USA reported an average bicyclist speed of 14.1 mph (22.7 km/h).[54] This study also reported an average deceleration rate of 7.5 ft./sec./sec. (2.3 m/sec./sec.). The AASHTO *Guide for the Development of Bicycle Facilities*[55] reports the following 98th percentile speeds for bicyclists: 17.6 ft./sec. (12 mph, 19.3 km/h) for advanced riders, 12.0 ft./sec. (8.2 mph, 13.2 km/h) for basic riders, and 9.1 ft./sec. (6.2 mph, 10.0 km/h) for young riders.

Section 9D.02 of the 2009 MUTCD[5] requires agencies to review and adjust signal timing on bikeways to consider the needs of bicyclists. The ITE *Traffic Control Devices Handbook*, 2nd Edition[12] states that signal timing at intersections should provide adequate time for bicyclists who enter the intersection legally at the end of the green phase to complete their crossing before conflicting traffic receives a green indication. The approach in the *Handbook* determines the yellow change interval in accordance with recommended practices based on motor vehicle speed. This interval should not typically be modified to accommodate bicyclists, as it could result in unpredictable effects on motor vehicle traffic.

The red clearance interval can be adjusted by an extension time, *e*, to provide any additional time for bicyclist clearance. However, the red clearance interval should not be excessively long; this could affect intersection capacity and progression, and could encourage drivers to enter the intersection after the end of the yellow change interval.

The following formula may be used to determine the crossing time for bicyclists making a rolling entry into an intersection during the green interval:

$$BCT_R = t + \frac{V}{2a} + \frac{W + L}{V} \tag{37}$$

Where:

BCT_R	=	Bicycle crossing time—rolling entry (sec.);
t	=	Perception-reaction time, typically 1 sec.;
V	=	Bicycle speed in intersection (ft./sec. or m/sec.), typically 14.7 ft./sec. (10 mph) or 4.5 m/sec. (16 km/h) (can be greater);
A	=	Bicycle deceleration rate—wet pavement (ft./sec./sec.), typically 5 ft./sec./sec. or 1.5 m/sec./sec.;
W	=	Intersection width (ft. or m); and
L	=	Bicycle length (ft. or m), typically 6 ft. or 2 m.

The value of BCT_R from this equation may then be used to determine the bicycle clearance time:

$$BCT_R \leq e + Y + \mathrm{R} \tag{38}$$

Where:

BCT_R	=	Bicycle crossing time—rolling start (sec.);
e	=	Extension time (sec.);
Y	=	Yellow change interval(s), typically 3 to 6 sec.; and
R	=	Red clearance interval(s), typically 0 to 6 sec.

The *Handbook* suggests that, if the calculated bicycle crossing time exceeds the maximum allowable values for yellow change plus red clearance intervals, consideration can be given to some type of adaptive signal timing triggered by bicycle detection.

With the widespread application of pedestrian countdown signals, there has been research to determine whether this additional information visible to drivers, affects their behavior. Study results have been mixed, some showing a definite impact on driver behavior, but there is not enough data available to draw a final conclusion. Eccles, Tao, and Magnum[56] evaluated the effect of pedestrian countdown signals on driver behavior in a before-after study of five intersections in Montgomery County, MD, USA. Observations found no difference in vehicle approach speeds during the pedestrian change interval.

Schattler, Wakim, Datta, and McAvoy[57] conducted a comparative study of 10 intersections in Peoria, IL, USA. Five intersections had pedestrian countdown signals, while five comparison intersections had only the traditional flashing “DON’T WALK” pedestrian signal indication. Researchers examined vehicle positions approaching or in the intersection during the yellow change interval and after the red signal indication was illuminated. Results supported the findings in

Eccles et al. indicating that drivers approaching intersections with countdown signals did not take greater risks.

Two smaller-scale studies, however, found pedestrian countdown signals did affect driver behavior.

Huey and Ragland[58] explored the effects in a study limited to one test intersection and one comparison intersection in Berkeley, CA, USA. Observations supported a difference in behavior when pedestrian countdown signals were present. At the intersection with pedestrian countdown signals, significantly fewer vehicles entered the intersection between the yellow and red signals. This intersection also had significantly fewer vehicles stop at the intersection. The authors noted that while fewer vehicles may enter during the change interval in the presence of pedestrian countdown signals, vehicles that do enter may travel at greater speeds.

Huey and Ragland noted that while fewer vehicles may enter during the change interval in the presence of pedestrian countdown signals, vehicles that do enter may travel at greater speeds.

Schrock and Bundy[59] studied four intersections along a single arterial in Lawrence, KS, USA, two of which had pedestrian countdown signals, and two of which had only the flashing "DON'T WALK" pedestrian signal indication. Researchers divided driver behavior into five categories:
1) driver decelerated at or after the onset of yellow and stopped;
2) driver decelerated before the onset of yellow and stopped;
3) driver continued normally through the intersection; 4) driver accelerated through the intersection; and 5) driver ran the red light to continue through the intersection. The compared results support the hypothesis that pedestrian countdown signals have an effect on driver behavior. Drivers approaching intersections with pedestrian countdown signals appeared to drive less aggressively than those approaching intersections with only the flashing "DON'T WALK" pedestrian signal indication.

Current Practice

Respondents were asked what policies agencies have for unusual cases and what data are collected to time change intervals. A small number of respondents, 5 of 267 (2 percent), reported having policies for heavy vehicle traffic. Four respondents (1 percent) also indicated their agencies use heavy vehicle traffic volume data. Comments addressed heavy vehicle traffic approach speed and deceleration abilities. One comment noted comfortable deceleration and approach speed for a transit vehicle with standing passengers is much lower. A rate of 8 ft./sec./sec. (2.4 m/sec./sec.) was cited by several agencies as applicable for heavy trucks.

A minority of the respondents reported considering pedestrians and bicyclists. Three of 267 (1 percent) have policies on pedestrians, and 4 of 267 (1 percent) have policies addressing bicyclists. Only one respondent reported the agency considers the presence of bicycle lanes. Few agencies also collect pedestrian and bicyclist data: twenty-seven collect pedestrian volumes, three collect pedestrian characteristic data, and one collects bicyclist volume data.

Recommendation

With any special user group or special conditions where there are more detailed data and supporting information available to the engineer, the engineer should consider the information and appropriately apply it to the situation. Heavy vehicles and transit vehicles may decelerate more slowly, but may also travel at less than the 85th percentile speed. If alternate timings are considered based on this factor, all elements of the equations should be considered by the engineer.

If the roadway has been designated as a bicycle facility (such as a bike lane or bike route) the timing of the red clearance interval should consider the needs of bicyclists. Bicycle traffic has historically and traditionally not been separately addressed when determining change intervals. The literature review supported bicyclist speeds ranging from 9.1 ft./sec. (6.2 mph, 10.0 km/h) to 17.6 ft./sec. (12 mph, 19.3 km/h). The relatively low speeds of bicyclists should enable them to more easily stop prior to the stop line than motorists. However, in the case that a bicyclist continues through the intersection after the onset of yellow, the time needed to clear the intersection will likely be longer. For such a case, consideration should be given to adjusting the red clearance interval by an extension time to provide additional time for intersection clearance by bicyclists. As with all modifications for unique conditions, this modification must be applied with engineering judgment.

Pedestrian change intervals should not affect the timing of vehicle phase change intervals. Pedestrian countdown signals may help inform drivers of when the signal indication will change. However, the literature supports that countdown signals do not affect approach speeds and driver behavior.

If necessary, the engineer may choose to accommodate unique road users by increasing the PRT, increasing the approach speed for the yellow change interval, reducing the approach speed for the red clearance interval, reducing the deceleration rate, and/or increasing the vehicle length. The resulting change intervals should not be shorter than the intervals calculated based on the typical assumptions for each of the variables.

2.16 Special Road Conditions

Special conditions may include, but are not limited to, the following:

- atypical traffic operations;
- closely spaced intersections;
- high-volume, uncontrolled driveways within the indecision zone and the intersection;
- skewed intersections;
- multi-leg intersections;
- atypical roadway geometry;
- locations at or near highway rail crossings; and
- locations with adverse weather for an extended period of time.

Literature

There is limited research specifically focusing on each of these issues.

Current Practice

Respondents were asked if their agency used any special considerations for skewed intersections, railroad crossings, pre-emption technology, advance warning signals, adverse weather, varying speeds by time of day, pavement conditions, special events, and other unique traffic conditions.

> *Approximately one-third of the responding agencies indicated they did not have a procedure for special road conditions.*

Specific conditions mentioned were adjustment to interval calculation for double left-turn movement versus single left turns, rounding up of minimum yellow change interval for a uniform value for the through phase on streets with a speed limit lower than 40 mph (64 km/h), turning-speed adjustment for cross street gutters, and adjustment to red clearance interval due to long paths through an intersection (e.g., large width, skewed intersection, single point urban interchange, etc.). Approximately one-third of the responding agencies indicated they did not have a procedure for special road conditions. Other agencies' staff applied engineering judgment to these instances and/or treated them on a case-by-case basis.

Recommendation

Engineers should exercise judgment by using the most appropriate application values in applying the kinematic equation in the alternate conditions which are applied to the travel path, approach, and passage speeds through the intersection. The engineers should document all their assumptions, data, methods, and findings when determining appropriate yellow change and red clearance intervals for unique conditions.

2.17 Implementation

Recommendation

The MUTCD[5] states that yellow change intervals must be predetermined and programmed in traffic signal controllers. Different intervals are allowable by timing plan but cannot change from cycle to cycle within the same timing plan as that would violate MUTCD guidance.

An important aspect of the development work of ITE is that all its standards and recommended practices are advisory only. ITE has no regulatory authority and does not enforce their use. All standards and recommended practices are used and/or applied on substantially public facilities and only have status when officially sanctioned by the governing agency. Their use by public agencies is usually in the interest of safeguarding the welfare and safety of the private users of the products or facilities themselves. Acceptance and implementation of these recommended practices by public agencies is at their sole discretion.

2.18 Safety

Literature

Numerous studies over the past 50 years have attempted to examine and quantify various safety effects associated with modifications to change interval timing and phasing. These studies generally fall into three categories: 1) effects of yellow change interval timing on red-light running and late exits, 2) effects of yellow change interval timing on crashes, and 3) crash effects associated with installing red clearance intervals. The quality and reliability of the results vary in these studies. This review attempted to identify all relevant and available reports, assess their quality, document references, and provide a synthesis of the methods and main results. In summary, despite the diversity of research methods and range of findings, the following general conclusions can be drawn from the available body of literature.

Effects of Change Interval Timing on Red-Light Running and Late Exits

At intersection approaches where yellow change interval durations are set below values associated with ITE formula or similar kinematic-based formulae, increasing yellow change interval duration to the ITE formula values can significantly reduce red-light running. Studies by Bonneson and Zimmerman,[60] Harders,[61] Munro and Marshall Associates,[62] Retting et al.,[63] Van der Horst,[64] and Wortman et al.[15] found increasing yellow duration by about 1 sec. at approaches deemed to have insufficient change interval timing was associated with reductions in red-light running ranging from about 36 to 90 percent. The best estimate of the impact of change interval timing on red-light running, based on the better-designed studies, is about a 36 to 50 percent reduction. Likewise, increasing yellow change and/or red clearance interval timing to achieve values associated with the ITE formula, or similar kinematic-based formulae, can significantly reduce late exits, as well as reductions in potential vehicle conflicts. Evidence in these studies generally shows increasing the duration of red clearance intervals does not increase red-light running.

Effects of Change Interval Timing on Crashes

Past studies reported a range of crash effects associated with modifications to change interval timing, reflecting differences in research methods, outcome measures, settings, specific types of modification to change interval timing, and other factors. Several crash-based studies report that setting change interval timing to values associated with the ITE formula is associated with reduced total crashes, injury crashes, and/or right-angle crashes. The best estimate of effect on crashes, based on leading before-after studies, is about an 8 to 14 percent reduction in total crashes, and about a 12 percent decrease in injury crashes. Some studies report evidence of increased risk of rear-end crashes when yellow interval duration is increased, which may reflect the increased decision-making time allotted to the motorist. Benioff et al.[13] concluded that excessively long yellow intervals "definitely are hazardous."

Several crash-based studies report that setting change interval timing to values associated with the ITE formula is associated with reduced total crashes, injury crashes, and/or right-angle crashes. The best estimate of effect on crashes is about an 8 to 14 percent reduction in total crashes, and about a 12 percent decrease in injury crashes.

Tables 5.5 and 5.6 of *NCHRP Report 705: Evaluation of Safety Strategies at Signalized Intersections*[65] show increasing the red clearance interval and total change period can reduce crash frequency. The report found that, if the yellow change plus red clearance interval is to a value exceeding that obtained from the kinematic equation, rear-end crashes were reduced 36 percent, though reductions in total and injury crashes were statistically insignificant. If the yellow change plus red clearance interval is increased but to a value less than that obtained from the kinematic equation, the report found injury crashes were reduced by 34 percent and total crashes by 27 percent, though reductions in rear-end crashes were statistically insignificant.

Table 14-7 of the *Highway Safety Manual*[66] supplies specific crash modification factors to different crash types based on modifying the yellow change and red clearance interval to the formulae provided in *Determining Vehicle Change Intervals: A Proposed Recommended Practice*.[1]

Crash Effects Associated with Installing Red Clearance Intervals

The red clearance interval has many supporters who believe it helps prevent right-angle crashes associated with red-light running. In addition, supporters claim the proper use and setting of the red clearance interval helps clear more "sneakers" during the change interval and reduces the chances of a protected left-turn phase from being warranted. Detractors argue that red clearance intervals simply encourage and reward red-light-running behavior. Unfortunately, crash-based research evaluations do not provide a clear consensus on the safety effects of installing red clearance intervals. Most available studies have relatively weak experimental designs and other limitations. Results vary from relatively large crash reductions, modest crash reductions, crash increases, to no effects. The most comprehensive study on this topic, conducted by Souleyrette et al.[46] (which still had methodological limitations), suggested modest short-term crash reductions, but no longer-term effects associated with installing red clearance intervals. Absent more definitive research, the crash effects of installing red clearance intervals are inconclusive.

Current Practice

Respondents were asked if longer yellow change intervals negatively affect driver behavior and safety. They commented that the recommended practice should prioritize safety over efficient operations. One member asked if there is existing literature on changing the distribution of the yellow change and red clearance intervals while keeping the total change interval constant.

Recommendation

Literature is not definitive on the long-term impact on driver behavior and safety of vehicle change intervals longer than the ITE formula, or similar kinematic-based formulae. Continued research in this area is necessary before conclusions can be drawn. The general consensus is that excessively long change intervals should be avoided to not only encourage driver compliance, but to also reduce impacts on intersection capacity and efficiency. To the best of the study team's knowledge, there is no published research on the impact of changing the distribution of the yellow change and red clearance intervals while keeping the total change period constant.

2.19 Driver Behavior

Literature

Driver behavior to change intervals may be influenced by driver, vehicle, and environmental characteristics. Driver characteristics may consist of the driver's age, gender, mental capacity, and experience. Vehicle characteristics may include the condition of the vehicle, vehicle type, vehicle features, and vehicle model. Environmental characteristics consider other external factors such as weather condition, time of day, traffic volume, road classification, number of lanes, surrounding land use, regional driving characteristics, and level or type of enforcement.

A 2005 study by FWHA asked focus group and survey participants how they would react to hypothetical traffic situations.[67] Specifically, focus group participants were provided graphics showing a car in front of the participant's car approaching an intersection with the following verbal description: "Approaching a signalized intersection at speed, the light turns yellow. The driver is far enough away from the intersection that he/she can stop if he/she brakes hard, but is likely to enter the intersection on an early red if he/she accelerates." The participants included 18 to 35 year old, 35 to 55 year old, and 65 year old and older drivers of both genders from Washington, DC, USA, Chicago, IL, USA, and Seattle, WA, USA. Based on their stated preferences, older drivers were more likely to stop at the yellow indication to avoid running a red light because stopping is their default driving behavior in this scenario, while middle-aged and younger drivers would run the red light. Unless middle-aged drivers thought the vehicle in front of them was going to stop, going through the light was their default strategy. Traffic and driving conditions, being in a rush, and the behaviors of a lead vehicle were all factors that led younger drivers to go through the light. Younger drivers were generally less likely to go through the light if their parents were in the car. For most drivers, additional factors that influence their behavior in this scenario include congestion levels, pedestrian activity, obstructions, cross traffic, and roadway conditions. The results also showed driver behavior is influenced by attitude, beliefs, and social norms.

Hicks, Tao, and Tabacek[68] conducted an observational study of driver behavior to change intervals. The study required researchers to observe drivers' characteristics and their decision to pass or stop after the onset of the yellow indication at intersections in Maryland. Preliminary observations indicated female and older drivers were more conservative in their stay-or-go decision than their male and younger counterparts, being less likely to enter the intersection during the yellow change interval. An ordered probit model regressed from this data by Xiang, Chou, Chang, and Tao[69] found a positive relationship between "aggressive" behavior and yellow change interval duration, intersection width, average flow speed, and traffic volume. The model also suggested drivers at major intersections with multiple lanes displayed more "conservative" behavior, and drivers of pick-up trucks and compact and subcompact vehicles displayed more "aggressive" behavior. Further extensions on this study by Liu, Chang, Hicks, and Tabacek[70] were able to classify drivers into three distinct groups based on their responses during the yellow change interval: aggressive, normal, and conservative. The authors identified the aggressive level of drivers based on a comparison of the speed of a vehicle approaching the intersection to the average flow speed. They concluded a driver's behavior during the yellow change interval is influenced by average traffic flow speed, green split timing, traffic volume, signal coordination, number of lanes, cell phone usage, vehicle type, driver age, and driver gender.

A 2008 experimental study by El-Shawarby, Amer, and Rakha[31] compared observed driver behavior to change intervals in a testing facility. The researchers concluded that older drivers' indecision zone had greater variance and were closer to the intersection than those of middle-aged and younger drivers. The findings additionally suggested female drivers were more likely to stop at the intersection after the onset of the yellow indication and had indecision zones closer to the intersection compared to male drivers.

2.20 Recommendations for Further Study

Yellow change and red clearance intervals have been a topic of research since 1960. The intent of this chapter is to capture the evolution of professional research, current practice, and consensus of the engineering community to define a recommended practice. Any identified item noted for additional study could be used to further refine this recommended practice as the results from professional research are completed and properly vetted. During the course of the development of the recommended practice by the technical committee and peer review panel, a number of topics were identified where additional study or new research would be helpful to expand the body of knowledge on this topic. The following topics were identified.

- *Approach and passage speed variations associated with different left-turn lane characteristics.* Left-turn lanes have a variety of geometric and operational characteristics potentially affecting their approach and passage speeds that would benefit from additional research including, for example, speed limits less than 30 mph (50 km/h), turn-lane length, number of lanes, signal phasing, and movements where U-turns are allowed in addition to left turns on single- or multi-lane approaches. This research should also examine the significance of these potential effects and whether they could be practically applied to the calculations.

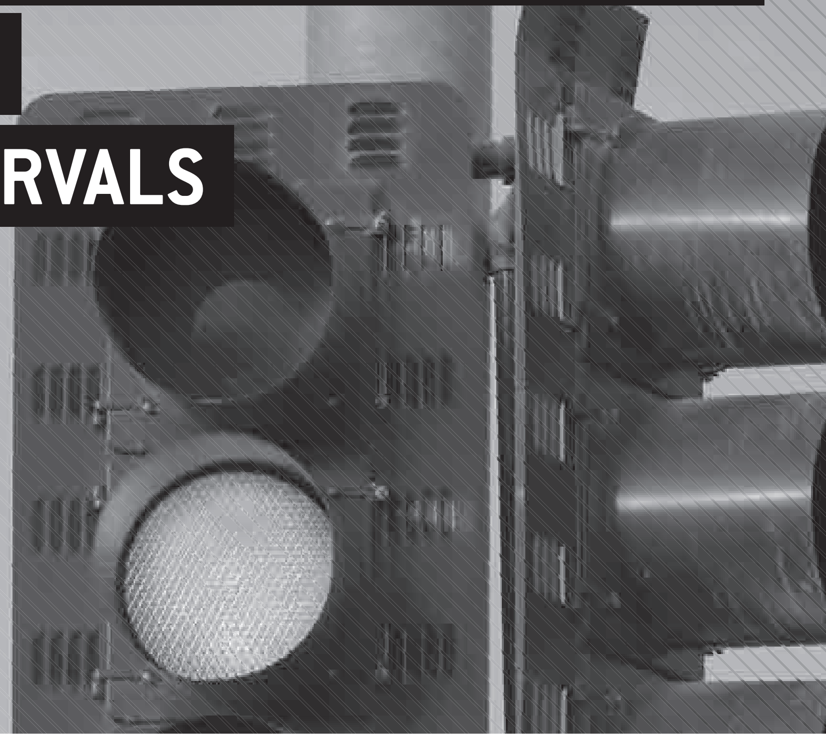
- *Approach and passage speed variations for different right-turn lane characteristics.* Right-turn lanes have a variety of geometric and operational characteristics potentially affecting the approach and passage speeds that would benefit from additional research including, for example, speed limits less than 30 mph (50 km/h), turn-lane length, number of lanes, signal phasing, and conflicting pedestrians. While characteristics of right turns are analogous to left turns, how they affect application of the equations may be different. This research should also examine the significance of these potential effects and whether they could be practically applied to the calculations.
- *Data collection methods for approach speeds of through movements compared to posted speed limits.* With the expansion of automated traffic signal performance measures programs (e.g., Utah DOT and Indiana DOT) the ability to collect and archive intersection detection data, including vehicles' speeds, is rapidly increasing. Supporting research would examine processes to use data from detector infrastructure to provide an expanded data set of approach speeds by lane, roadway classification, speed limit, under- and over-saturated traffic conditions, and area type.
- *Approach speeds on "non-posted" roadways.* There is need for development of supporting information to determine approach speeds for driveways, alleyways, short approaches, entrances to new developments, and other "non-posted" roadways. The proposed research should determine values and guidance for practical application for these types for roadways. Research should also examine the significance of these potential effects.
- *Passage speed variation on the path through an intersection from left or right turns.* The approach to estimating the passage speed for a turning path through an intersection in this recommended practice is based on the AASHTO horizontal curve design speed equation. Additional empirical analysis of field data in comparison to theoretical values for small radii and the curvature of complex paths, along with guidance for application, would enhance understanding of these relationships.
- *Easy to implement method to determine the length of travel path through intersections for turning movements and complex intersection geometries.* Vehicles making turning movements or moving through complex intersection geometries typically do not follow circular paths. Research should also examine the significance of these potential effects and whether they could be practically applied to the calculations.
- *Perception-reaction time and deceleration rate for alerted drivers for turning movements.* Additional data and analysis, for both right- and left-turning vehicles, of the effect of a planned choice of movement by an alerted driver on perception-reaction time and deceleration rate. Similarly, whether information from countdown pedestrian signal indications affect perception-reaction time and deceleration rate. The effect of different age groups, vehicle types, and approach speeds on these two parameters would need to be incorporated into the study.
- *Yellow change interval length in excess of calculated value.* The literature is not definitive on the long-term impact on driver behavior and safety of yellow change intervals longer than those calculated by the kinematic equation-based formulae. Continued research in this area would be helpful.
- *Effect of weather conditions.* Many jurisdictions implement special timing plans for inclement weather situations. An additional study opportunity could examine the significance of these potential effects and whether they could be practically applied to the formula or assumptions.
- *Safety benefits of yellow change and red clearance intervals.* Additional study of driver compliance rates with and their sensitivity to signal timings set for yellow change and red clearance intervals per recommended practice and/or other potential methods would be helpful. This work should incorporate left-, through- and right-turn movements as well as the impact on instances of red-light running.
- *Detectors.* Additional study would be useful on the effect of detector configuration in determining approach speeds in such cases as multi-detector designs for high-speed approaches, advance end-of-green warning, or dynamic red clearance extension.

Results of these potential research subjects should lead to easy-to-implement, practical methods for operating agencies.

Although these items have been identified for further study, this recommended practice captures the current, readily available research.

Chapter 3

RECOMMENDED METHOD FOR DETERMINING YELLOW CHANGE AND RED CLEARANCE INTERVALS



3.1 Approach

This chapter presents a proposed recommended practice of ITE for timing the yellow change and red clearance intervals for traffic signals. This practice is based on the recommendations found in Chapter 2 of this document and applies the kinematic equation-based formula to calculate yellow change and red clearance intervals.

Agencies are encouraged to adopt a policy for establishing the method to calculate yellow change and red clearance intervals and to apply it consistently throughout their jurisdiction. Significant road-user benefit is derived by design consistency.

3.2 Definitions

The yellow change interval is the duration of the steady yellow signal indication following every circular green, green arrow, flashing yellow arrow, or flashing red arrow signal indication displayed during the operation of a traffic signal in steady mode. The purpose of the yellow change interval is to warn traffic of an impending change in right-of-way assignment

The red clearance interval is the duration of the steady red signal indication following the steady yellow signal indication which is displayed to potentially conflicting traffic movements at a traffic signal. The purpose of the red clearance interval is to provide additional time for a vehicle legally in the intersection before conflicting traffic movements begin.

3.3 General Requirements and Considerations

The following general requirements apply to the determination of yellow change and red clearance intervals based on Section 4D.26 of the 2009 MUTCD[5] and recommendations from the state-of-the-practice review:

1. *The duration of the yellow change interval and red clearance interval shall be determined using engineering practices.*
2. *The durations of yellow change intervals and red clearance intervals shall be consistent with the determined values within the technical capabilities of the controller unit.*
3. *The duration of a yellow change interval shall not vary on a cycle-by-cycle basis within the same signal timing plan.*

4. *Except as provided below in items a to c below, the duration of a red clearance interval shall not be decreased or omitted on a cycle-by-cycle basis within the same signal timing plan.*
   a. *The duration of a red clearance interval may be extended (increased) from its predetermined value for a given cycle based upon the detection of a vehicle that is predicted to violate the red signal indication.*
   b. *When an actuated signal sequence includes a signal phase for permissive/protected (lagging) left-turn movements in both directions, the red clearance interval may be shown during those cycles when the lagging left-turn signal phase is skipped and may be omitted during those cycles when the lagging left-turn signal phase is shown.*
   c. *The duration of a yellow change interval or a red clearance interval may be different in different signal timing plans for the same controller unit.*

Section 4D.26 of the 2009 MUTCD[5] provides the following guidance on minimum and maximum yellow change and red clearance intervals:

> *"A yellow change interval should have a minimum duration of 3 seconds and a maximum duration of 6 seconds. The longer intervals should be reserved for use on approaches with higher speeds.*
>
> *"Except when clearing a one-lane, two-way facility…or when clearing an exceptionally wide intersection, a red clearance interval should have a duration not exceeding 6 seconds."*

Uniformity of Intervals

Uniform yellow change intervals can reduce user confusion about the duration of change intervals. If yellow change intervals for concurrently terminating phases differ, apply yellow change intervals greater than the minimum calculated value for the approach. Uniform change intervals may be implemented along corridors or arterials and in coordinated systems.

Minimums and Maximums

The minimum value for the yellow change interval is 3.0 sec. and the maximum value is 6.0 sec. The maximum value for yellow change interval may be modified with engineering judgment and/or study to address special road conditions. The minimum value of the red clearance interval is 1.0 sec.

Rounding

Calculated values ending in 0.01 to 0.09 shall be rounded up to nearest 0.1 sec.

3.4 Formula for Calculating Change and Clearance Intervals

The kinematic equations for calculating the yellow change and red clearance intervals with the approach speed input in mph and a unit conversion factor applied are as follows:

$$Y = t + \frac{1.47V_{85}}{2a + 64.4g} \qquad \text{(A)}$$

$$R = \left[\frac{W + L}{1.47V_{85}}\right] - t_s \qquad \text{(B)}$$

Where:

V_{85} = 85th percentile approach speed (mph);
1.47 = unit conversion factor to convert ft./sec. to mph;
a = deceleration rate (ft./sec./sec.);
g = approach percent grade, in percent divided by 100;
W = width of intersection (ft.);
L = length of vehicle (ft.); and
t_s = conflicting movement start up delay (sec.).

The kinematic equations for calculating the yellow change and red clearance intervals with the approach speed input in km/h and a unit conversion factor applied are as follows:

$$Y = t + \frac{0.28V}{2a + 19.6g} \qquad \text{(Metric units) (C)}$$

$$R = \left[\frac{W + L}{0.28V}\right] - t_s \qquad \text{(Metric units) (D)}$$

Where:

Y = yellow change interval (sec.);
t = perception-reaction time (sec.);
V_{85} = 85th percentile approach speed (km/h);
a = deceleration rate (m/sec./sec.);
g = grade of approach (percent/100, downhill is negative grade);
R = red clearance interval (sec.);
W = width of intersection, stop line to far-side no-conflict point (m);
L = length of vehicle (m); and
t_s = conflicting movement start-up delay (sec.).

The equations for the yellow change interval, Equations A and C, provide the minimum yellow change interval required to allow time for the motorist to see the yellow signal indication and decide whether to stop or to enter the intersection. This time includes the motorist's perception-reaction time, generally 1.0 sec. It then allows time for motorists that are too close to the intersection to decelerate comfortably to a stop with enough time to travel the stopping distance and thus reach the intersection before the right-of-way terminates. The equations for the red clearance interval, Equations B and D, allow motorists that enter the intersection before the yellow change interval terminates time to continue through to the far side of the intersection before conflicting traffic enters. These times are dependent on the characteristics of the traffic and the roadway environment. If there is a grade on the approach to the intersection, Equations A and C adjust the time to account for the gravitational acceleration caused by the slope of the road and its impact on the braking distance that must be traversed.

3.5 Application for Through Movements

This section presents information on calculating the change and clearance intervals for through movements at a signalized intersection. Values for the inputs to the kinematic equation are provided. The engineer may collect field values as necessary and apply them to these equations for intersections for a variety of operating characteristics. If the engineer collects field measurements to modify the inputs to the equation, the measurements should be taken during representative conditions. Appendix C, Table C.1 and Table C.2 provide example calculations.

Perception-Reaction Time, t

The perception-reaction time is a minimum 1.0 sec. PRT of 1.0 sec. is sufficient for most users; however, if local conditions, driving population age, or a supporting engineering study suggest a value higher than 1.0 sec. is appropriate, engineering judgment may be used to modify this value upward.

85th Percentile Approach Speed, V_{85}

The approach speed is the 85th percentile approach speed as determined under free-flow conditions, if known or as determined by a speed study. The engineer can collect the 85th percentile free-flow speed in the field using a number of methods including those in ITE's *Traffic Engineering Handbook*, and ITE's *Manual of Transportation Engineering Studies*. Data to support engineering studies to determine an 85th percentile approach speed can be collected by various methods, including RADAR, LIDAR, paired loop detectors, microwave detectors, and other tools.

Further, please note that while the 2009 MUTCD[5] does not allow cycle-by-cycle changes in yellow change interval time (line 09 Section 4D.26), the engineer has an option that, "The duration of a yellow change interval or a red clearance interval may be different in different signal timing plans for the same controller unit." (line 13 Section 4D.26). Should the engineer choose to use this option, free-flow approach speed should be measured for the period associated with each signal timing plan.

If the 85th percentile speed is unavailable and a speed study is not conducted, the 85th percentile approach speed for through movements may be estimated by the following equations for calculating the yellow change interval:

$$V_{85}\text{ (through)} = SL + 7 \qquad \textit{(U.S. units)} \text{ (E)}$$

Where:
V_{85} = 85th percentile speed, in mph; and
SL = posted speed limit, in mph.

$$V_{85}\text{ (turn)} = SL + 11 \qquad \textit{(Metric units)} \text{ (F)}$$

Where:
V_{85} = 85th percentile speed (km/h); and
SL = posted speed limit (km/h).

Prior to implementing this alternate estimation method for 85th percentile approach speed, an agency should consider the applicable speed limit laws and its speed limit engineering process. An agency implementing this approach should document the policy decision and applicable context of the roadway's characteristics and classification, available resources, and the need for engineering judgment.

For through movements, the prevailing speed of vehicles clearing the intersection during the red clearance interval is assumed to be the same as the 85th percentile approach speed. If speed studies demonstrate a different speed through the intersection along the vehicle path, the engineer should use judgment to apply the new primary data to the calculation. This may be necessary if the intersection is used regularly by bicyclists, has complex geometry, and the engineer determines a red clearance interval based on prevailing speed is not sufficient for the intersection.

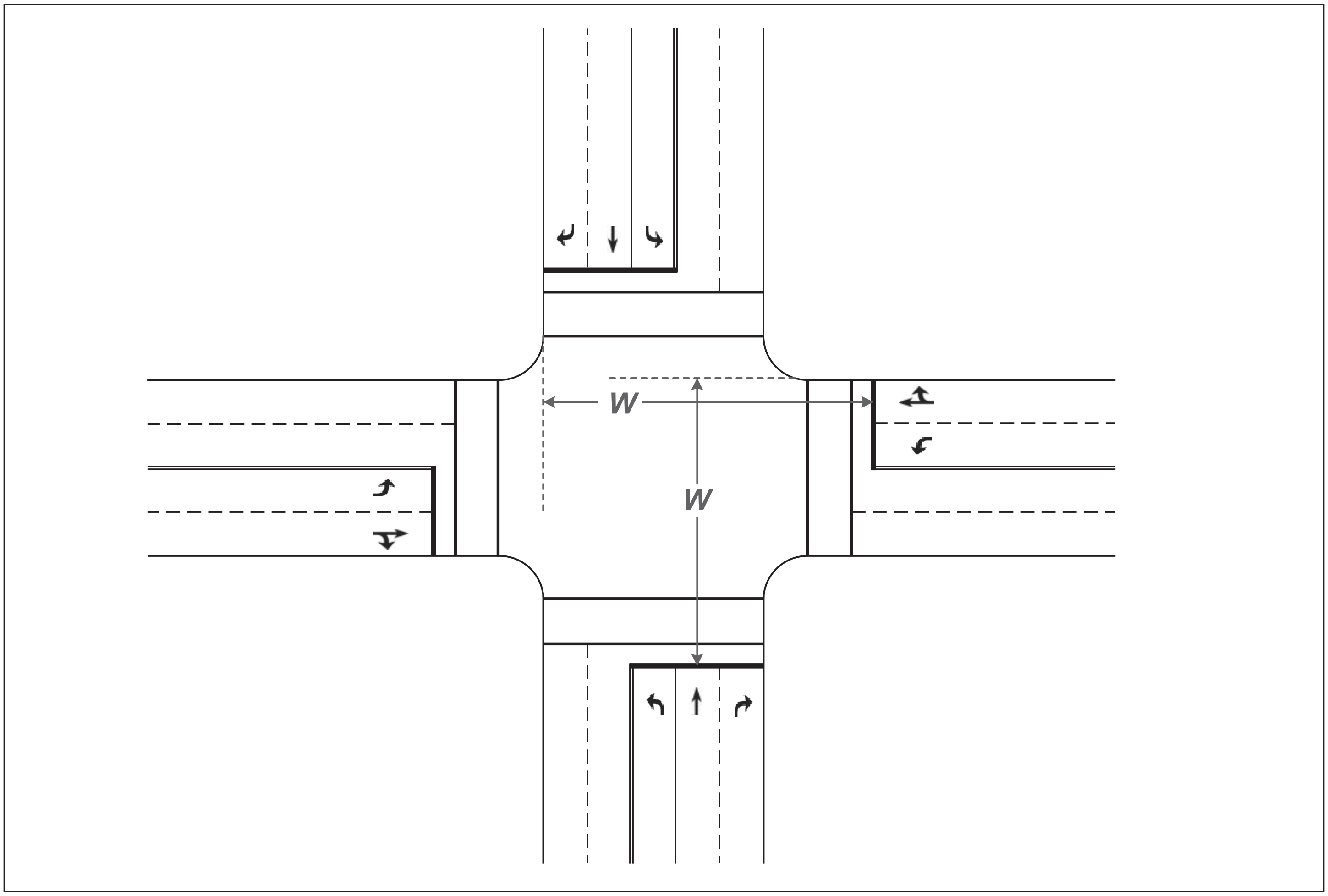
Deceleration Rate, a

The deceleration rate is 10 ft./sec./sec. (3 m/sec./sec.). Deceleration rate of 10 ft./sec./sec. (3 m/sec./sec.) is appropriate for most users; however, if local conditions, vehicle type, driving population age, or a supporting engineering study suggest a different value is appropriate, engineering judgment may be used to modify this value.

Approach Grade, g

Approach grades are determined at the upper boundary of the indecision zone based on the 85th percentile approach speed and applied to all movements on that approach. The approach grade is negative for downgrades and positive for upgrades. For existing intersections, approach grade is best estimated based on field conditions, and/or as-built design plans confirmed with field observations. For new intersections, approach grade can be obtained from design plans.

Figure 3.1: Diagram of Intersection Width Measurement for Through Movements



Width of Intersection, W

Intersection width is the total distance from the stop bar to the curb-line extension, or outside edge of the travel lane, of farthest conflicting movement along the vehicle's travel path. Figure 3.1 illustrates intersection width for through movements. The curb-line extension, rather than the far-side crosswalk, if any, is generally recommended as the far point of the conflicting movement. This reference point is used, since the average pedestrian entry design time is 3 sec., during which a clearing vehicle will have traveled far beyond a crosswalk adjacent to the far-side curb extension. Field measurements with an apparatus of choice provide the most accurate road width measure distance. However, as-built design plans, recent aerial photography, GPS, and surveys that reflect the current layout of the intersection enable practitioners to gather measurements of intersection width with minimal resources or field work safety concerns.

Vehicle Length, L

The vehicle length is 20 ft. (6.1 m). The engineer can use a longer vehicle length if 20 ft. is not representative of vehicles using the intersection. Longer vehicle length may be considered based on supporting vehicle classification study and application of engineering judgment.

Conflicting Movement Start-Up Delay, t_s

Conflicting movement start-up delay is 1.0 sec. A 1.0 sec intersection entry delay factor is subtracted from the calculated red clearance interval as long as the result is not less than 1.0 sec. Higher intersection entry delay values may be used based on engineering judgment or as supported by an engineering study.

3.6 Application for Turning Movements

Left-Turn Applications

This section presents information on calculating the change and clearance intervals for left-turn movements at a signalized intersection. **Values for the inputs to the kinematic**

equation for left-turn movements are the same as through movements for

- **deceleration rate, *a*, as 10 ft./sec./sec. (3.0 m/sec./sec.)**
- **approach grade, *g*, as determined at the upper boundary of the indecision zone based on the 85th percentile approach speed and applied to all movements on that approach.**
- **vehicle length, *L*, as 20 ft. (6.1 m)**
- **conflicting movement start-up delay, t_s, as 1.0 sec.**

Values for the inputs to the kinematic equation for left-turn movements for approach speed and intersection width are different. The engineer may collect field values as necessary and apply them to these equations for intersections for a variety of operating characteristics. If the engineer collects field measurements to modify the inputs to the equation, the measurements should be taken during representative conditions. Appendix C Table C.3 and Table C.4 provide example calculations.

Perception-Reaction Time, *t*

The perception-reaction time is a minimum 0.6 sec. for left-turn movements. PRT of 0.6 sec. is sufficient for an alerted driver intending to make a left turn; however, if local conditions, driving population age, or a supporting engineering study suggest a value higher than 0.6 sec. is appropriate, engineering judgment may be used to modify this value upward.

85th Percentile Approach Speed, V_{85}

The approach speed for the yellow change interval is the 85th percentile approach speed for left-turning vehicles as determined under free-flow conditions, if known or as determined by a speed study. If the 85th percentile approach speed for the left-turn movement is unavailable and a speed study is not conducted, the 85th percentile approach speed for turning movements may be estimated as the speed limit minus 5 mph (8 km/h) by the following equation for calculating the yellow change interval:

$$V_{85}\,(\text{turn}) = SL - 5 \qquad (\textit{U.S. units})\ (\text{G})$$

Where:

V_{85} = 85th percentile speed (mph); and
SL = posted speed limit (mph).

$$V_{85}\,(\text{turn}) = SL - 8 \qquad (\textit{Metric units})\ (\text{H})$$

Where:

V_{85} = 85th percentile speed (km/h); and
SL = posted speed limit (km/h).

Prior to implementing this alternate estimation method for 85th percentile approach speed, an agency should consider the applicable speed limit laws and its speed limit engineering process. An agency implementing this approach should document the policy decision and applicable context of the roadway's characteristics and classification, available resources, and the need for engineering judgment.

For left-turn movements, the prevailing speed of vehicles clearing the intersection along the turning path during the red clearance interval is assumed to be an 85th percentile speed of 20 mph (32.2 km/h). If more speed studies demonstrate a different speed through the intersection along the vehicle path, the engineer should use judgment to apply new primary data to the calculation. This may be necessary if the intersection is used regularly by bicyclists, has sharp turning radii, and the engineer determines a red clearance interval based on prevailing speed is not sufficient for the intersection.

Width of Intersection, *W*

Intersection width is the total distance from the stop bar to the curb-line extension, or outside edge of the travel lane, of farthest conflicting movement along the vehicle's travel path. Figure 3.2 illustrates intersection width for left-turn movements. Where there are multiple lanes present, either on the approach or departure legs of the intersection, the longest natural turning path should be used. The curb-line extension, rather than the far-side crosswalk, if any, is generally recommended as the far point of the conflicting movement. This reference point is used, since the average pedestrian entry design time is 3 sec., during which a clearing vehicle will have traveled far beyond a crosswalk adjacent to the far-side curb extension. Field measurements and verification of turning path with an apparatus of choice provide the most accurate road width measure distance. However, as-built design plans, recent aerial photography, GPS, and surveys that reflect the current layout of the intersection enable practitioners to gather measurements of intersection width with minimal resources or field work safety concerns.

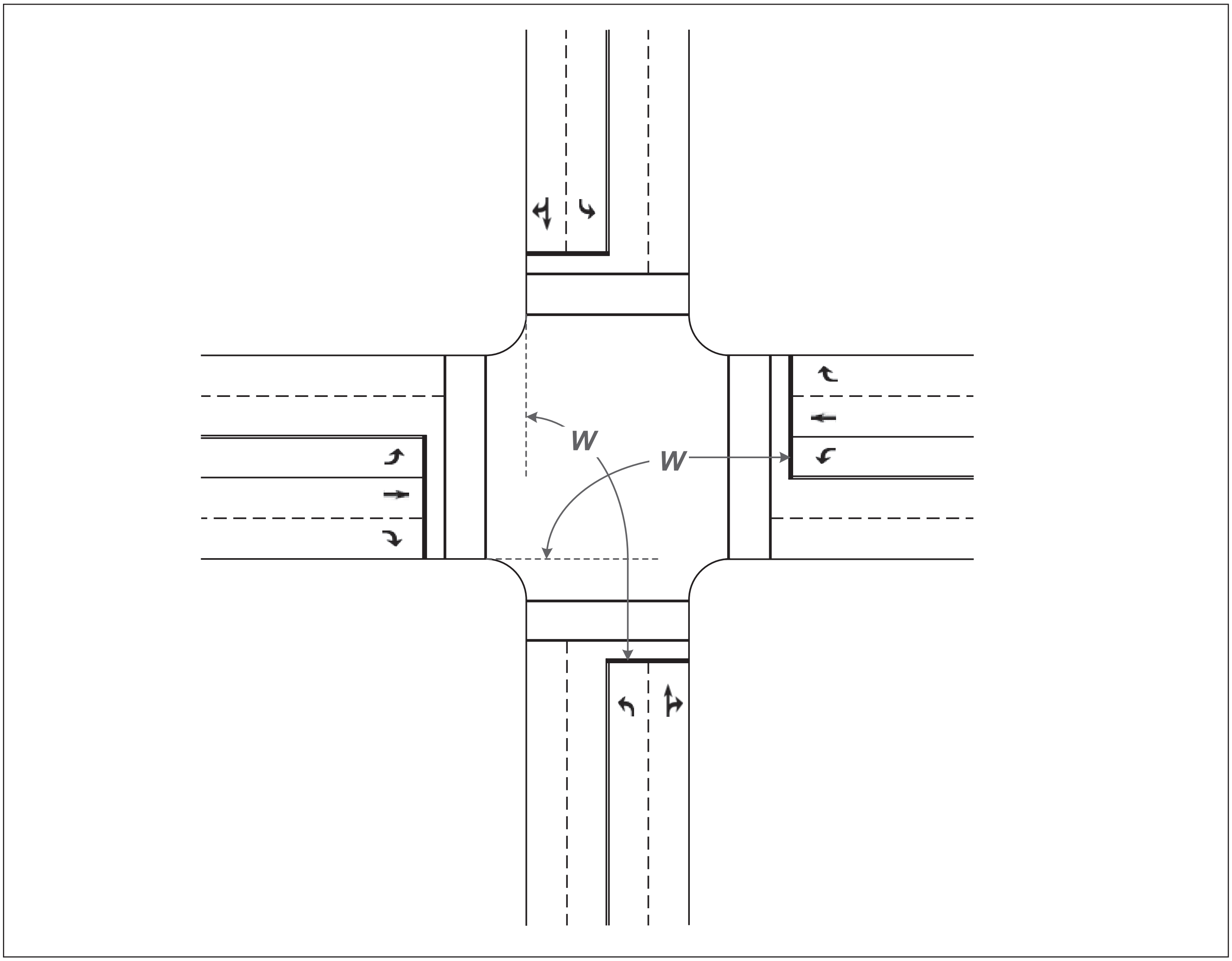
Right-Turn Applications

When the termination of a right-turn signal indication occurs with the termination of a signal indication of an adjacent movement on the same approach, the yellow change and red clearance intervals are recommended to be the same duration as the adjacent movement.

Signal Phasing

The following notes the recommended approach to calculating the yellow change and red clearance intervals for different types of signal phasing.

Figure 3.2: Diagram of Intersection Width Measurement for Left-Turn Movements



Protected-Only Applications

Protected-only left-turn movements: calculate the yellow change and red clearance intervals for each approach and implement as calculated. The intervals can be of different duration than opposing approaches or adjacent through-movement phase.

Permissive-Only Applications

Permissive-only left-turn movements: calculate the yellow change and red clearance intervals for opposing approaches, including through movements, and use the longest of the calculated values (left, through, or combination). The intervals should be the same duration for the left-turn and through movements on opposing approaches to ensure that termination is concurrent.

Protected/Permissive Applications

Protected/permissive left-turn movements: calculate the yellow change and red clearance intervals and implement as described above for the respective protected and permissive portions of the phase. The implemented yellow change and red clearance intervals should be the longer of the calculated values for the left-turn and through movement phases. The intervals should be the same duration for the left-turn and through movement phases on opposing approaches to ensure that termination is concurrent.

3.7 Special Considerations

Wide Intersections

Using the formulas, the engineer can calculate values for wider intersections. For very wide intersections, this will result in long change intervals. Engineers should use their engineering judgment in the application of these intervals. In addition to calculating the time needed for the through vehicle to clear the intersection, the engineer should also calculate the time needed for any concurrent left turns to clear the intersection. For wide intersections, the time for concurrent lefts may be greater than the time for through movements because of the slower speed of the left turns. The intersection width for turning vehicles should be measured from the stop bar along the vehicle's path to the farthest point of conflicting traffic which includes vehicles and pedestrians. The engineer may use 20 mph (32.2 km/h) as the speed of the turning vehicle or can collect the average turning vehicle speed in the field.

Bicycle Traffic

Bicycles have different operating characteristics than other vehicles and wide intersections can be problematic due to the time necessary to traverse the distance. If a roadway has been designated as a bicycle facility (such as a bike lane), consideration should be given to adjusting the red clearance interval by an extension time to provide additional time for bicyclists to clear the intersection before conflicting traffic. The engineer may decide to add an extension of time to the red clearance interval, depending on the bicycle speed, length, deceleration, crossing distance, and the judgment of the engineer, to accommodate the clearance needs of bicyclists who enter the intersection at the end of the yellow. The combined value of the red clearance interval plus extension should not exceed the maximum allowable value.

3.8 Measures of Effectiveness

Yellow Change Interval

The primary measure of effectiveness for the yellow change interval length is the percentage of vehicles entering the intersection after the termination of the yellow indication—that is, during the succeeding red indication. Another measure of effectiveness is the percent of cycles in a traffic signal timing plan where vehicles were observed entering the intersection during the red clearance interval. However, prevailing regional practices may influence driver behavior and may make comparisons difficult.

The logic behind the methodology for determining the length of the yellow change interval is that the duration of the yellow change interval should provide a reasonable driver—that is too close to the intersection to stop safely and comfortably—with adequate time to traverse the distance and legally enter the intersection before the signal turns red or right of way terminates. The yellow indication is not meant to cover the time to comfortably stop, as part of the stopping maneuver can safely occur during the red indication. A reasonable driver closer to the intersection will proceed into and through the intersection when presented with a yellow indication. A reasonable driver farther away from the intersection at the onset of the yellow indication will decide to stop and has sufficient distance to do so comfortably. Values used for the variables in the equation are selected to determine the time for the non-stopping driver traveling at the prevailing speed to traverse the stopping distance based on the mean reaction time and deceleration of drivers in the indecision zone when the light turns yellow.

When the percentage of vehicles that entered on a red indication exceeds that which is locally acceptable, the yellow change interval may be lengthened until the percentage conforms to desirable standards.

Red Clearance Interval

As with the yellow change interval, the test of a red clearance interval length is whether the desired result is produced. Do vehicles clear the area of conflict, as defined by the equation's intent and an identified desirable compliance percentage? What is the percent of cycles where vehicles failed to clear the intersection during the red clearance interval? If the yellow change interval length is too short, vehicles will still be in the area of conflict even if the red clearance interval length is correct. Therefore, the yellow change interval length should be evaluated first.

Many of the factors that affect the yellow change interval length, particularly vehicle mix, may also affect the red clearance interval length. The presence of a large percentage of trucks or bicycles in the traffic stream may change the speed range.

3.9 Monitoring and Evaluation

The selected yellow change and red clearance interval durations, once established and implemented, should be maintained in official records with other supporting documentation of traffic signal timing. These official records should include information about the traffic signal including the signal design, signal timing, and date when the timings were implemented. These official records should be used to track changes made to the signal timing at an intersection.

Review of traffic signal yellow change and red clearance interval durations should be part of an agency's traffic signal program management plan. Traffic signal program management plans identify operational objectives and associated performance measures that are further defined though processes and procedures. Such reviews ensure the values still adequately reflect the conditions at the intersection and the characteristics of the traffic. Factors that result in the need to review and adjust traffic signal timing may include

- Changes in traffic demand since the intersection was last timed. This could include changes in side-street demand, turning-movement volume or spill back, main-street demand, or vehicle

mix (for example, a higher percentage of trucks). Changes in vehicle demand could also be reflected in general increases in demand that cause the need for longer periods with peak period timing.
- Changes in intersection operations (for example, addition of an approach lane or the moving of a bus stop from near side to far side) that influence the need for timing.
- Changes in pedestrian traffic due to land use changes (for example, the opening of a residence for the elderly which requires longer pedestrian clearance times) or the need for handicapped features.
- Changes to agency policies or national standards, such as the *Manual on Uniform Traffic Control Devices*.
- Temporary changes in roadway operations due to construction.
- Observations of previously unnoticed conditions by an alert motorist or staff member, or through use of a traffic management center.
- Changes in vehicle, bicycle, and pedestrian safety data and supporting analysis.
- Agreements with other jurisdictions to coordinate with their signal systems, or to provide coordinated response to incidents on parallel facilities.

The *Traffic Signal Timing Manual*[11] includes additional information on signal timing programs and underlying processes. If revisions are necessary based on any of the above factors, they should be addressed in a timely manner and revisions, consistent with the procedures in an agency's program, should be recorded.

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Glossary

15th Percentile Speed—The speed at which 15 percent of the vehicles in a sample are traveling at or below.

85th Percentile Speed—The speed at which 85 percent of the vehicles in a sample are traveling at or below.

95th Percentile Speed—The speed at which 95 percent of the vehicles in a sample are traveling at or below.

All-Red Interval or All-Red Clearance Interval—An out-of-date term for the interval following the yellow change interval and preceding the next conflicting green interval during which **all** traffic at an intersection view a red signal indication and are not permitted in the intersection. This term has been replaced by "red clearance interval" because "all-red" is too specific to one combination of signal indications and does not adequately describe the interval appropriately with complex signal phasing techniques.

Amber Light—A term describing the yellow signal indication.

Amber Light Phase—The duration of the yellow signal indication.

Approach Grade—The slope of the roadway at the entrance, or approach, to an intersection.

Approach Speed—The velocity of a vehicle approaching an intersection.

Change Interval—A term used to describe the first interval following the green or flashing arrow interval during which the steady yellow signal indication is displayed. The current approach, for clarity, is to describe this term as the "yellow change interval."

Change Period—Refers to the period of time between conflicting green signal indications; may consist of a yellow interval only or a yellow change and red clearance interval.

Conflicting Traffic Movements—Traffic movements that, if allowed into the intersection at the same time, would intersect paths.

Cycle—One complete sequence of all traffic signal indications.

Dilemma Zone—The theoretical location in advance of a traffic signal where a driver is presented with the condition of a yellow signal indication and a choice to stop prior to entering the intersection or to go through the intersection. Mathematically, at a given travel time from the stop line, a "dilemma" is defined as existing if there is both a non-zero probability of stopping before the intersection and a non-zero probability of going through the intersection. This definition is also known as a Type I Dilemma Zone. Based on the assumed parameters and the appropriate application of the kinematic equation to determine the change period, this type of dilemma zone does not exist.

Engineering Judgment—The evaluation of available pertinent information, and the application of appropriate principles, provisions, and practices as contained in professional documents and other sources, for the purpose of deciding upon the applicability, design, operation, or installation of a traffic control device. Engineering judgment can be exercised by an engineer, or by an individual working under the supervision of an engineer, through the application of procedures and criteria established by the engineer. Documentation of engineering judgment is not required but is helpful to support to decisions made.

Engineering Study—The comprehensive analysis and evaluation of available pertinent information, and the application of appropriate principles, provisions, and practices as contained in professional documents and other sources, for the purpose of deciding upon the applicability, design, operation, or installation of a traffic control device. An engineering study is performed by an engineer, or by an individual working under the supervision of an engineer, through the application of procedures and criteria established by the engineer. An engineering study is documented.

Entering the Intersection—Crossing the stop line or, if none exists, crossing the nearest edge of a crosswalk threshold or, if neither exists, crossing the near-side conflicting curb line.

Green Interval—A period of time indicating that vehicles are displayed a green signal indication.

Green Signal Indication—The illumination of the green traffic signal lens during which vehicular traffic facing a circular green signal may proceed straight through or turn right or left, unless a sign at such place prohibits either such turn. But vehicular traffic, including vehicles turning right or left, shall yield the right of way to other vehicles and pedestrians lawfully within the intersection or an adjacent crosswalk at the time such signal is exhibited.

Indecision Zone—This term has to do with the probabilistic behavior of a driver in response to the choice of stopping or going when shown a yellow signal indication. It can be defined as the location between the distance at which 90 percent of the drivers would stop and the distance at which 10 percent of the drivers would stop. The indecision zone typically extends from a travel time of about 2.5 sec. to 5 sec. in advance of the intersection for the prevailing speed of traffic. This zone exists at the beginning of yellow indication regardless of duration of the yellow clearance interval. The term is also referred to as a Type II dilemma zone.

Interval—the part of a signal cycle during which signal indications do not change.

Intersection—intersection is defined as follows:

a. The area embraced within the prolongation or connection of the lateral curb lines or, if none, the lateral boundary lines of the roadways of two highways that join one another at, or approximately at, right angles, or the area within which vehicles traveling on different highways that join at any other angle might come into conflict.

b. The junction of an alley or driveway with a roadway or highway shall not constitute an intersection, unless the roadway or highway at said junction is controlled by a traffic control device.

c. If a highway includes two roadways that are 30 ft. or more apart (a median), every crossing of each roadway of such divided highway by an intersecting highway shall be a separate intersection.

d. If both intersecting highways include two roadways that are 30 ft. or more apart, every crossing of any two roadways of such highways shall be a separate intersection.

e. At a location controlled by a traffic control signal, regardless of the distance between the separate intersections as defined in (c) and (d) above.

    1. If a stop line, yield line, or crosswalk has not been designated on the roadway (within the median) between the separate intersections, the two intersections and the roadway (median) between them shall be considered as one intersection;

    2. Where a stop line, yield line, or crosswalk is designated on the roadway on the intersection approach, the area within the crosswalk and/or beyond the designated stop line or yield line shall be part of the intersection; and

    3. Where a crosswalk is designated on a roadway on the departure from the intersection, the intersection shall include the area extending to the far side of such crosswalk.

Kinematic Equation—An equation based on the aspects of motion apart from considerations of mass and force.

Overlap—Signal timing technique that provides a way to operate a particular movement with one or more phases.

Perception-Reaction Time—The time needed for a motorist to see the signal indication (perception) and then begin executing the appropriate response (reaction).

Brake-Response Time—The time needed for a motorist to see the signal indication (perception), execute the appropriate response (reaction), and the vehicle response to the input (onset of braking as evidenced by vehicle brake lights). This is a common field-measured estimate of perception-reaction time.

Permissive Yellow Law—Describes local laws that allow vehicles to enter the intersection throughout the entire yellow change interval, and be in the intersection during the red indication as long as they entered the intersection during the yellow change interval.

Phase—The entire sequence of green, yellow, and red intervals in a cycle assigned to an independent traffic movement or combination of movements.

Phase-Change Interval—Refers to the period of time between conflicting green signal indications. May consist of a yellow change interval only or a yellow change and a red clearance interval.

Reasonable Driver—A term used to describe the typical motor vehicle operator who executes roadway maneuvers in a safe and prudent manner, demonstrates rational driving behavior, and responds appropriately to road conditions and traffic control devices. A reasonable driver who is close to the intersection stop bar at the onset of the yellow traffic signal indication will proceed into and through the intersection, while a reasonable driver farther from the intersection stop bar will decide to stop if she or he has sufficient distance to do so comfortably.

Red Clearance Interval—An interval following the yellow change interval and preceding the next conflicting green interval during which all conflicting traffic movements at an intersection view a red signal indication and are not permitted to enter the intersection. It allows time for vehicles which entered the intersection during the yellow change interval to exit, or clear, the intersection.

Red Signal Indication—The illumination of the red traffic signal lens during which traffic movements facing the lens are not permitted to enter the intersection.

Restrictive Yellow Law—Describes local laws that do not allow vehicles to be in the intersection during the red indication, even if they entered the intersection during the yellow interval.

Right-of-Way—The precedence of passage of a traffic movement into an intersection over other traffic movements at that intersection.

Signal Timing—The distribution of a length of time (cycle) between traffic movements including the allocation of green, yellow, and red indications for each movement.

Signal Indication—The illumination of a traffic signal lens.

Signal Lens—That part of the signal section that redirects the light coming directly from the light source and its reflector, if any.

Posted Speed Limit—The maximum (or minimum) travel speed on a street established by law, ordinance, or regulation.

Stop Line or Stop Bar—A pavement marking that denotes where traffic should stop in advance of an intersection.

Stopping Distance—The distance a vehicle travels while decelerating to a complete stop.

Traffic Movements—Describes the combination of vehicles, bicycles, and pedestrians at an intersection grouped together by the direction in which they are traveling through the intersection.

Traffic Signal—A power-operated traffic control device by which traffic is warned or directed to take a specific action. Traffic is warned or directed by a series of green, yellow, and red lenses that illuminate.

Velocity—The speed and direction that a vehicle is traveling.

Warning Clearance Interval—An antiquated term that refers to the yellow change interval.

Warning Interval—An antiquated term that refers to the yellow change interval.

Yellow Interval—An outdated term used to describe the first interval following the green or flashing arrow interval during which the steady yellow signal indication is displayed.

Yellow Change Interval—The first interval following the green or flashing arrow interval during which the steady yellow signal indication is displayed.

Yellow Clearance Interval—An incorrect term to describe the yellow change interval.

Yellow Signal Indication—The illumination of the yellow signal lens.

Yellow Warning Indication—An antiquated term that refers to the yellow signal indication.

Appendix A
SURVEY OF PRACTICE

Traffic Signal Change Intervals Survey

The Institute of Transportation Engineers (ITE) is in the process of preparing Guidelines for Determining Traffic Signal Change Intervals: An ITE Recommended Practice (RP). In 1985 ITE published a Proposed Recommended Practice entitled *Determining Vehicle Change Intervals* that was not ratified to become an RP. Later, in 2001, ITE published the informational report *A History of the Yellow and All-Red Intervals for Traffic Signals*. In the interim, changes in technology, automated enforcement, the availability of new primary data, further research and the public and professional concern that a defined standard of reference does not exist with regard to this topic have led to the initiative to develop this RP.

This survey of transportation agencies is part of the effort to determine the current state-of-the-practice and to provide the user with an overview of key considerations to determine yellow change and red clearance intervals for traffic signals and their application. Results from this survey will be provided to the research team preparing the National Cooperative Highway Research Program document entitled *Guidelines for Timing Yellow and All-Red Intervals at Traffic Signals* as well.

Because this survey is intended to specifically target public agencies, we ask that responses be submitted only by public agency employees. Thank you.

START SURVEY!

Traffic Signal Change Intervals Survey

Questions marked with an asterisk (*) are mandatory.

1 * Please identify the location of your agency.

- United States
- Canada
- Other, please specify

2 State / Province

Survey Questions

3 Does your agency have a formal policy for timing the traffic signal changes intervals?

- Yes
- No

4 Is there a formal policy for the use of the optional all-red interval?

- Yes
- No

5 If yes to either question, please submit material via email to dnoble@ite.org with the subject line of "TSCI Survey"

Note: An email address will need to be provided.

Name:

Agency:

Address 1:

Address 2:

City/Town:

State/Province:

Zip/Postal Code:

Country:

Email Address:

6 If there is no formal policy, generally what method do you use to determine the duration of change intervals:

- The following kinematic equation is used: $CP = t + V / (2a+64.4g) + (W+L) / V$
- A uniform value is used for all intersections (e.g. 4 seconds).
- A uniform value is used for all intersections (e.g. 4 seconds), except where conditions warrant an exception to the uniform timing.
- A table of values by approach speed is applied to all intersections.
- Other, please specify

7 What, if any, are your minimum and maximum values for the yellow intervals, all-red intervals, and total change interval?

Yellow min	
Yellow max	
All-red min	
All-red max	
Total interval min	
Total interval max	

8 If you use the kinematic equation displayed in question 2, how do you allocate time between the yellow and all-red interval?

- The calculated value from the first two terms of the equation is allocated to the yellow interval and the third term is allocated to the all-red interval.
- The yellow interval is set at a uniform duration (e.g., four seconds) and the remainder is allocated to the all-red interval.
- The all-red interval is set at a uniform duration (e.g., one second) and the remainder is allocated to the

yellow interval.

- The entire time is allocated to the yellow interval. The all-red interval is not used.
- Other, please specifiy

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9 If you use an equation similar to the kinematic equation in question 2, what values do you use for the following variables:

Perception reaction time (*t*) =

Deceleration (*a*) =

Vehicle length (L) =

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10 If speed is used to calculate the interval durations, what speed do you use?

- 85th percentile approach speed
- Posted speed limit
- Design speed
- Other, please specify

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11 If a different speed is used to calculate the all-red interval, what speed do you use (for example, some agencies used 85th percentile speed to time the yellow interval and posted speed to time the all-red interval)?

- 85th percentile approach speed
- Posted speed limit
- Design speed
- Other, please specify

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12 If speed measurements are collected in the field, how frequently are they updated?

- Not collected.
- Only once to time the interval.
- Annually

- As conditions change
- Other, please specify

13 Other than speed, do you collect any field measurements (e.g., intersection width, pedestrian volumes) prior to timing the change interval?

14 Do you have a procedure for special situations (e.g. left or right turn signals) or for special populations (e.g. large trucks, bicyclists, transit vehicles with standing passengers)?

15 Comments or additional information.

SUBMIT

Appendix B

DEFINITIONS OF YELLOW SIGNAL INDICATION FOR VEHICLES BY STATE AND PROVINCE

Table B.1: Definitions of Yellow Signal Indication for Vehicles by State and Province

UNITED STATES	
States	**Definition of Yellow Signal indication (for vehicles)**
Alabama (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
Alaska (P)	*No specific information available, assume Uniform Vehicle Code as default.*
Arizona (P)	Vehicular traffic facing a steady yellow signal is warned by the signal that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Arkansas (P)	Vehicular traffic facing the signal is warned that the red or "STOP" signal will be exhibited immediately thereafter, and vehicular traffic shall not enter the intersection when the red or "STOP" signal is exhibited
California (P)	A driver facing a steady circular yellow or yellow arrow signal is, by that signal, warned that the related green movement is ending or that a red indication will be shown immediately thereafter.
Colorado (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
Connecticut	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter, when vehicular traffic shall stop before entering the intersection unless so close to the intersection that a stop cannot be made in safety
Delaware (P)	Vehicular traffic facing the circular yellow signal is thereby warned that a red signal for the previously permitted movement will be exhibited immediately thereafter.
Florida (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Georgia (P)	Traffic, except pedestrians, facing a steady CIRCULAR YELLOW or YELLOW ARROW signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection
Hawaii (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Idaho (P)	A driver facing a steady circular yellow or yellow arrow signal is being warned that the related green movement is ending, or that a red indication will be shown immediately after it.
Illinois (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
Indiana (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is warned that the related green movement is being terminated and that a red indication will be exhibited immediately thereafter.
Iowa*	A "steady circular yellow" or "steady yellow arrow" light means vehicular traffic is warned that the related green movement is being terminated and vehicular traffic shall no longer proceed into the intersection and shall stop. If the stop cannot be made in safety, a vehicle may be driven cautiously through the intersection.
Kansas (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Kentucky (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Louisiana	Vehicular traffic facing a steady yellow signal alone is thereby warned that the related green signal is being terminated or that a red signal will be exhibited immediately thereafter and such vehicular traffic shall not enter or be crossing the intersection when the red signal is exhibited.

Note:
(P) indicates *permissive* law state
* States allowing intersection entry and clearance in circumstances where it is unsafe or not possible to stop are generally not in conflict with the permissive yellow law.
No notation indicates a restrictive law state (Louisiana, Tennessee, Rhode Island, and West Virginia).

UNITED STATES	
States	**Definition of Yellow Signal indication (for vehicles)**
Maine (P)	If steady and circular or an arrow, means the operator must take warning that a green light is being terminated or a red light will be exhibited immediately
Maryland (P)	Vehicular traffic facing a steady yellow signal is warned that the related green movement is ending or that a red signal, which will prohibit vehicular traffic from entering the intersection, will be shown immediately after the yellow signal.
Massachusetts (P)	*No specific information available, assume Uniform Vehicle Code as default.*
Michigan*	If the signal exhibits a steady yellow indication, vehicular traffic facing the signal shall stop before entering the nearest crosswalk at the intersection or at a limit line when marked, but if the stop cannot be made in safety, a vehicle may be driven cautiously through the intersection.
Minnesota (P)	Vehicular traffic facing a circular yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection, except for the continued movement allowed by any green arrow indication simultaneously exhibited.
Mississippi*	Vehicular traffic facing the signal shall stop before entering the nearest crosswalk at the intersection, but if such stop cannot be made in safety a vehicle may be driven cautiously through the intersection.
Missouri (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection
Montana (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is warned that the traffic movement permitted by the related green signal is being terminated or that a red signal will be exhibited immediately thereafter. Vehicular traffic may not enter the intersection when the red signal is exhibited after the yellow signal.
Nebraska*	Vehicular traffic facing a steady yellow indication is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection, and upon display of a steady yellow indication, vehicular traffic shall stop before entering the nearest crosswalk at the intersection, but if such stop cannot be made in safety, a vehicle may be driven cautiously through the intersection
Nevada (P)	Vehicular traffic facing the signal is thereby warned that the related green movement is being terminated or that a steady red indication will be exhibited immediately thereafter, and such vehicular traffic must not enter the intersection when the red signal is exhibited.
New Hampshire (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
New Jersey*	Amber, or yellow, when shown alone following green means traffic to stop before entering the intersection or nearest crosswalk, unless when the amber appears the vehicle or street car is so close to the intersection that with suitable brakes it cannot be stopped in safety. A distance of fifty feet from the intersection is considered a safe stopping distance for a speed of twenty mph, and vehicles and street cars if within that distance when the amber appears alone, and which cannot be stopped with safety, may proceed across the intersection or make a right or left turn unless the turning movement is specifically limited.
New Mexico (P)	Vehicular traffic facing the signal is warned that the red signal will be exhibited immediately thereafter and the vehicular traffic shall not enter the intersection when the red signal is exhibited except to turn as hereinafter provided
New York (P)	Traffic, except pedestrians, facing a steady circular yellow signal may enter the intersection; however, said traffic is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
North Carolina (P)	When a traffic signal is emitting a steady yellow circular light on a traffic signal controlling traffic approaching an intersection or a steady yellow arrow light on a traffic signal controlling traffic turning at an intersection, vehicles facing the yellow light are warned that the related green light is being terminated or a red light will be immediately forthcoming.
North Dakota (P)	Vehicular traffic facing a steady circular yellow or yellow arrow indication is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic may not enter the intersection.

Note:
(P) indicates *permissive* law state
* States allowing intersection entry and clearance in circumstances where it is unsafe or not possible to stop are generally not in conflict with the permissive yellow law.
No notation indicates a restrictive law state (Louisiana, Tennessee, Rhode Island, and West Virginia).

UNITED STATES	
States	**Definition of Yellow Signal indication (for vehicles)**
Ohio (P)	Vehicular traffic, streetcars, and trackless trolleys facing a steady circular yellow or yellow arrow signal are thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic, streetcars, and trackless trolleys shall not enter the intersection.
Oklahoma (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
Oregon*	A driver facing a steady circular yellow signal light is thereby warned that the related right of way is being terminated and that a red or flashing red light will be shown immediately. A driver facing the light shall stop at a clearly marked stop line, but if none, shall stop before entering the marked crosswalk on the near side of the intersection, or if there is no marked crosswalk, then before entering the intersection. If a driver cannot stop in safety, the driver may drive cautiously through the intersection.
Pennsylvania (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green indication is being terminated or that a red indication will be exhibited immediately thereafter.
Rhode Island	Vehicular traffic facing the signal is warned by it that the red or “stop” signal will be exhibited immediately afterwards, and the vehicular traffic shall not enter or be crossing the intersection when the red or “stop” signal is exhibited.
South Carolina (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
South Dakota (P)	Vehicular traffic facing the signal is thereby warned that the red or “stop” signal will be exhibited immediately thereafter and such vehicular traffic shall not enter the intersection when the red or “stop” signal is exhibited.
Tennessee	Vehicular traffic facing the signal is warned that the red or “Stop” signal will be exhibited immediately thereafter and that vehicular traffic shall not enter or cross the intersection when the red or “Stop” signal is exhibited
Texas (P)	An operator of a vehicle facing a steady yellow signal is warned by that signal that: (1) movement authorized by a green signal is being terminated; or (2) a red signal is to be given.
Utah (P)	The operator of a vehicle facing a steady circular yellow or yellow arrow signal is warned that the allowable movement related to a green signal is being terminated.
Vermont (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green signal is being terminated or that a red signal will be exhibited immediately thereafter, when vehicular traffic shall not enter the intersection.
Virginia*	Steady amber indicates that a change is about to be made in the direction of the moving of traffic. When the amber signal is shown, traffic which has not already entered the intersection, including the crosswalks, shall stop if it is not reasonably safe to continue, but traffic which has already entered the intersection shall continue to move until the intersection has been cleared. The amber signal is a warning that the steady red signal is imminent.
Washington (P)	Vehicle operators facing a steady circular yellow or yellow arrow signal are thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection. Vehicle operators shall stop for pedestrians who are lawfully within the intersection control area as required by *RCW 46.61.235(1)*
West Virginia	Vehicular traffic facing the signal is thereby warned that the red or “stop” signal will be exhibited immediately thereafter and such vehicular traffic shall not enter or be crossing the intersection when the red or “stop” signal is exhibited.
Wisconsin*	When shown with or following the green, traffic facing a yellow signal shall stop before entering the intersection unless so close to it that a stop may not be made in safety.
Wyoming (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter

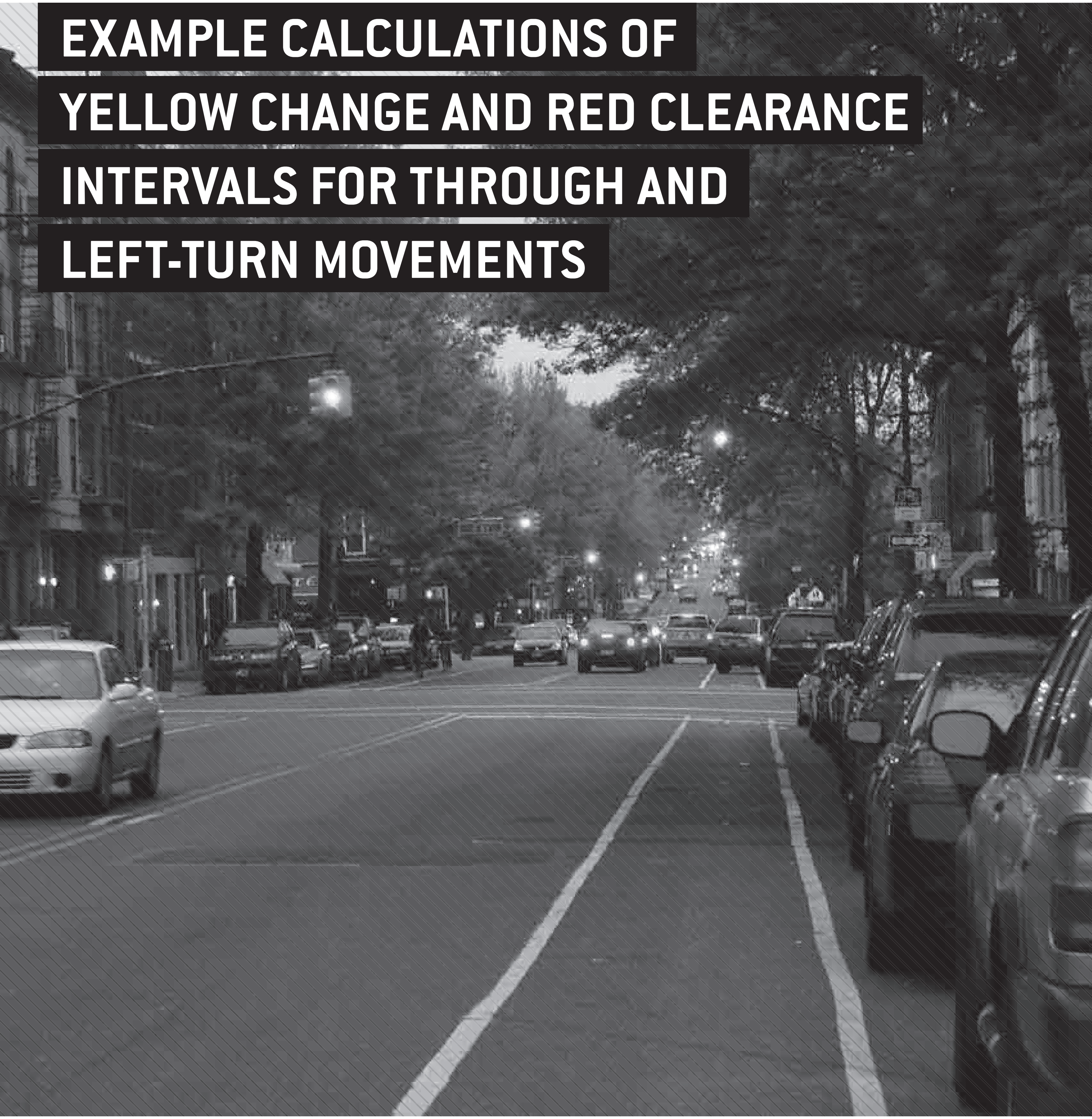
Note:
(P) indicates *permissive* law state
* States allowing intersection entry and clearance in circumstances where it is unsafe or not possible to stop are generally not in conflict with the permissive yellow law.
No notation indicates a restrictive law state (Louisiana, Tennessee, Rhode Island, and West Virginia).

CANADA	
Provinces	**Definition of Yellow Signal indication (for vehicles)**
Alberta	When a green light changes to yellow, it warns that the light will change to red immediately and drivers must prepare to stop or clear the intersection. Drivers approaching an intersection with a solid (not flashing) yellow traffic control light must bring their vehicles to a complete stop before the stop line or crosswalk, unless a point has been reached at the intersection where stopping cannot be done safely. If there is no stop line or crosswalk, vehicles must stop before the intersection. Drivers already in the intersection and facing a yellow light must safely clear the intersection.
British Columbia	When a yellow light alone is exhibited at an intersection by a traffic control signal, following the exhibition of a green light, (a) the driver of a vehicle approaching the intersection and facing the yellow light must cause it to stop before entering the marked crosswalk on the near side of the intersection, or if there is no marked crosswalk, before entering the intersection, unless the stop cannot be made in safety, (b) a pedestrian facing the yellow light must not enter the roadway, and (c) a pedestrian proceeding across the roadway and facing the yellow light exhibited after he or she entered the roadway (i) must proceed to the sidewalk as quickly as possible, and (ii) has the right of way for that purpose over all vehicles.
Ontario	Every driver approaching a traffic control signal showing a circular amber indication and facing the indication shall stop his or her vehicle if he or she can do so safely, otherwise he or she may proceed with caution.
Quebec	Unless otherwise directed by a sign or signal, when facing an amber light, the driver of a road vehicle or any person riding a bicycle must stop his vehicle before the pedestrian crosswalk or stop-line or, if none, before the near side of the roadway he is about to cross, unless he has entered it or is so close to it that he could not stop in safety; he may proceed only when a signal shows he may do so.
Manitoba	When a yellow or amber traffic control light or arrow is being shown at an intersection by a traffic control signal following or accompanying a green traffic control light, (a) the driver of a vehicle at or approaching the intersection and facing the light or arrow shall not enter the intersection, unless he can leave it before a red traffic control light or such other signal as next follows, begins to be shown;
New Brunswick	Except when otherwise directed by a peace officer, drivers and pedestrians shall obey the instructions exhibited by a traffic control signal exhibiting the words “Go”, “Passez”, “Caution”, “Attention”, or “Stop”, “Arrêt”, or exhibiting different coloured lights successively, one at a time or in combination or with arrows, in accordance with the following provisionsƐ: (b) yellow or amber alone or “Caution”, “Attention”, when shown immediately following the green or “Go”, “Passez”, signal, (i) the driver of a vehicle facing the signal is thereby warned that the red or “Stop”, “Arrêt”, signal will be exhibited immediately thereafter, and such driver shall not enter the intersection unless he is so close thereto that it is impossible to stop before so entering, and (ii) a pedestrian facing the signal is thereby warned that there is insufficient time to cross the roadway in safety, and if he starts to cross he shall yield the right-of-way to all vehicles.
Newfoundland and Labrador	Where a yellow or amber light alone is shown at an intersection by a traffic-control signal following a green light (a) the driver of a vehicle approaching the intersection and facing the yellow or amber light shall stop the vehicle at a clearly marked stop line or, if none, then immediately before entering the crosswalk on the near side of the intersection or, where there is no crosswalk, then immediately before entering the intersection, unless a stop cannot be made in safety; and (b) notwithstanding paragraph (a), the driver of a vehicle approaching the intersection and facing the yellow or amber light and intending to turn right at the intersection may, unless a traffic-control device prohibits a right turn to be made on a yellow or amber light, with caution, proceed and turn right at the intersection, but only after yielding the right-of-way to a pedestrian referred to in paragraph (7)(b) and to a vehicle proceeding in the intersection.
Northwest Territories	A driver facing a yellow or amber light as shown at an intersection by a traffic light shall stop his or her vehicle before it enters the intersection, unless a stop cannot be made in safety.
Nova Scotia	Yellow or amber light—all traffic facing this signal shall stop before entering an intersection at the place marked or the nearest side of the crosswalk but not past the signal unless the stop cannot be made in safety.
Nunavut	Not available
Prince Edward Island	When a yellow or amber light alone is shown at an intersection by a traffic-control signal following a green light signal, (a) the driver of a vehicle approaching the intersection and facing the light shall stop the vehicle at a clearly marked stop line or, if none, then immediately before entering the crosswalk on the near side of the intersection or, if there is no crosswalk then immediately before entering the intersection unless a stop cannot be made in safety.
Saskatchewan	If a traffic light at an intersection displays only an amber light, (a) the driver of a vehicle facing the light shall stop at the crosswalk, but, if the vehicle cannot be brought to a stop with safety, the driver may drive cautiously proceed through the intersection; and (b) pedestrians facing the light shall not enter the intersection.
Yukon Territories	When a yellow light is shown at an intersection by a traffic control signal at the same time as or following the showing of a green light, the driver of a vehicle approaching the intersection and facing the yellow light shall stop before entering (a) the marked crosswalk on the near side of the intersection; or (b) if there is no such marked crosswalk, then before entering the intersection, unless such a stop cannot be made in safety.

Note:
(P) indicates *permissive* law state
* States allowing intersection entry and clearance in circumstances where it is unsafe or not possible to stop are generally not in conflict with the permissive yellow law.
No notation indicates a restrictive law state (Louisiana, Tennessee, Rhode Island, and West Virginia).

Appendix C

EXAMPLE CALCULATIONS OF YELLOW CHANGE AND RED CLEARANCE INTERVALS FOR THROUGH AND LEFT-TURN MOVEMENTS



THROUGH MOVEMENTS

Yellow Change Interval

Table C.1: Example Calculation of Through Movement Yellow Change Intervals For Various Approach Speeds and Grades

Posted Speed Limit (mph)	85th Percentile Approach Speed (mph)	Grade (%)				
		-4	-2	0	2	4
Speed limits set by 85th percentile approach speed or speed zone survey						
25	25	3.2	3.0	3.0[a]	3.0[a]	3.0[a]
30	30	3.6	3.4	3.3	3.1	3.0
35	35	4.0	3.8	3.6	3.5	3.3
40	40	4.4	4.2	4.0	3.8	3.7
45	45	4.8	4.6	4.4	4.2	4.0
50	50	5.3	5.0	4.7	4.5	4.3
55	55	5.7	5.4	5.1	4.8	4.6
60	60	6.1[b]	5.8	5.5	5.2	5.0
If 85th percentile approach speed is unknown or a speed study is unavailable						
25	32	3.7	3.6	3.4	3.3	3.1
30	37	4.2	4.0	3.8	3.6	3.5
35	42	4.6	4.3	4.1	3.9	3.8
40	47	5.0	4.7	4.5	4.3	4.1
45	52	5.4	5.1	4.9	4.6	4.4
50	57	5.8	5.5	5.2	5.0	4.8
55	62	6.3[b]	5.9[b]	5.6	5.3	5.1
60	67	6.7[b]	6.3[a]	6.0	5.7	5.4
Posted Speed Limit (km/h)	**85th Percentile Approach Speed (km/h)**	**Grade (%)**				
		-4	**-2**	**0**	**2**	**4**
Speed limits set by 85th percentile approach speed or speed zone survey						
40	40	3.2	3.0	3.0[a]	3.0[a]	3.0[a]
50	50	3.7	3.5	3.4	3.2	3.1
60	60	4.3	4.0	3.8	3.7	3.5
70	70	4.8	4.5	4.3	4.1	3.9
80	80	5.3	5.0	4.8	4.6	4.4
90	90	5.9	5.5	5.2	5.0	4.8
100	100	6.0[b]	6.0	5.7	5.4	5.2
If 85th percentile approach speed is unknown or a speed study is unavailable						
40	51	3.8	3.6	3.4	3.3	3.2
50	61	4.3	4.1	3.9	3.7	3.6
60	71	4.9	4.6	4.4	4.2	4.0
70	81	5.4	5.1	4.8	4.6	4.4
80	91	5.9	5.6	5.3	5.0	4.8
90	101	6.0[b]	6.0[b]	5.8	5.5	5.2
100	111	6.0[b]	6.0[b]	6.0[b]	5.9	5.6

NOTE: Yellow change intervals calculated using 85th percentile approach speed, a perception-reaction time of 1.0 sec., and a comfortable deceleration rate of 10 ft./sec./sec. (3 m/sec./sec.).

a. The 2009 Edition of the MUTCD with Revision Numbers 1 and 2 incorporated,[5] dated May 2012, recommends a minimum duration of 3.0 sec. for the yellow change interval.

b. The 2009 Edition of the MUTCD with Revision Numbers 1 and 2 incorporated,[5] dated May 2012, recommends a maximum duration of 6.0 sec. for the yellow change interval.

Red Clearance Interval

Table C.2: Example Calculation of Through Movement Red Clearance Interval for Various Approach Speeds and Intersection Widths

Posted Speed Limit (mph)	85th Percentile Approach Speed (mph)	Width of Intersection (ft.)				
		30	50	70	90	110
Speed limits set by 85th percentile approach speed or speed zone survey						
25	25	1.0	1.0	1.5	2.0	2.6
30	30	1.0	1.0	1.1	1.5	2.0
35	35	0.0	1.0	1.0	1.2	1.6
40	40	0.0	1.0	1.0	1.0	1.3
45	45	0.0	1.0	1.0	1.0	1.0
50	50	0.0	0.0	1.0	1.0	1.0
55	55	0.0	0.0	1.0	1.0	1.0
60	60	0.0	0.0	1.0	1.0	1.0
If 85th percentile approach speed is unknown or a speed study is unavailable						
25	32	1.0	1.0	1.0	1.4	1.8
30	37	0.0	1.0	1.0	1.1	1.4
35	42	0.0	1.0	1.0	1.0	1.2
40	47	0.0	1.0	1.0	1.0	1.0
45	52	0.0	0.0	1.0	1.0	1.0
50	57	0.0	0.0	1.0	1.0	1.0
55	62	0.0	0.0	0.0	1.0	1.0
60	67	0.0	0.0	0.0	1.0	1.0
Posted Speed Limit(km/h)	**85th Percentile Approach Speed (km/h)**	**Width of Intersection (m)**				
		9.1	**15.2**	**21.3**	**27.4**	**33.5**
Speed limits set by 85th percentile approach speed or speed zone survey						
40	40	1.0	1.0	1.5	2.0	2.6
50	50	1.0	1.0	1.0	1.4	1.9
60	60	0.0	1.0	1.0	1.0	1.4
70	70	0.0	1.0	1.0	1.0	1.1
80	80	0.0	0.0	1.0	1.0	1.0
90	90	0.0	0.0	1.0	1.0	1.0
100	100	0.0	0.0	0.0	1.0	1.0
If 85th percentile approach speed is unknown or a speed study is unavailable						
40	51	1.0	1.0	1.0	1.4	1.8
50	61	0.0	1.0	1.0	1.0	1.4
60	71	0.0	1.0	1.0	1.0	1.0
70	81	0.0	0.0	1.0	1.0	1.0
80	91	0.0	0.0	1.0	1.0	1.0
90	101	0.0	0.0	0.0	1.0	1.0
100	111	0.0	0.0	0.0	1.0	1.0

NOTE: Based on an 85th percentile approach speed, an entry delay of 1.0 second, and an average vehicle length of 20 ft.(6.1 m).

Table C.3: Example Calculation of Left-Turn Movement Yellow Change Intervals

Posted Speed Limit (mph)	85th Percentile Approach Speed (mph)	Grade (%)				
		-4	-2	0	2	4
Speed limits set by 85th percentile approach speed or speed zone survey						
25	25	3.0[a]	3.0[a]	3.0[a]	3.0[a]	3.0[a]
30	30	3.2	3.0	3.0[a]	3.0[a]	3.0[a]
35	35	3.6	3.4	3.2	3.1	3.0[a]
40	40	4.0	3.8	3.6	3.4	3.3
45	45	4.4	4.2	4.0	3.8	3.6
50	50	4.9	4.6	4.3	4.1	3.9
55	55	5.3	5.0	4.7	4.4	4.2
If 85th percentile approach speed is unknown or a speed study is unavailable						
25	32	3.0[a]	3.0[a]	3.0[a]	3.0[a]	3.0[a]
30	37	3.0[a]	3.0[a]	3.0[a]	3.0[a]	3.0[a]
35	42	3.2	3.0	3.0[a]	3.0[a]	3.0[a]
40	47	3.6	3.4	3.2	3.1	2.9
45	52	4.0	3.8	3.6	3.4	3.3
50	57	4.4	4.2	4.0	3.8	3.6
55	62	4.9	4.6	4.3	4.1	3.9
Posted Speed Limit (km/h)	**85th Percentile Approach Speed (km/h)**	**Grade (%)**				
		-4	**-2**	**0**	**2**	**4**
Speed limits set by 85th percentile approach speed or speed zone survey						
40	40	3.0[a]	3.0[a]	3.0[a]	3.0[a]	3.0[a]
50	50	3.3	3.1	3.0	3.0[a]	3.0[a]
60	60	3.9	3.6	3.4	3.3	3.1
70	70	4.4	4.1	3.9	3.7	3.5
80	80	4.9	4.6	4.4	4.2	4.0
90	90	5.5	5.1	4.8	4.6	4.4
100	100	6.0	5.6	5.3	5.0	4.8
If 85th percentile approach speed is unknown or a speed study is unavailable						
40	32	3.0[a]	3.0[a]	3.0[a]	3.0[a]	3.0[a]
50	42	3.0[a]	3.0[a]	3.0[a]	3.0[a]	3.0[a]
60	52	3.4	3.2	3.1	3.0[a]	3.0[a]
70	62	4.0	3.7	3.5	3.4	3.2
80	72	4.5	4.2	4.0	3.8	3.6
90	82	5.1	4.7	4.5	4.2	4.0
100	92	5.6	5.2	4.9	4.7	4.4

NOTE: Yellow change intervals calculated using 85th percentile approach speed, a perception-reaction time of 0.6 sec., and a comfortable deceleration rate of 10 ft./sec./sec. (3 m/sec./sec.).

a. The 2009 Edition of the MUTCD with Revision Numbers 1 and 2 incorporated,[5] dated May 2012, recommends a minimum duration of 3.0 sec. for the yellow change interval.

Red Clearance Interval

Table C.4: Example Calculation of Left-Turn Movement Red Clearance Intervals

Turning Path Speed	Width of Intersection (ft.)				
	30	50	70	90	110
(mph)	Red Clearance Interval (sec.)				
20	1.0	1.4	2.1	2.8	3.5
Turning Path Speed	Width of Intersection (m)				
	9.1	15.2	21.3	27.4	33.5
(km/h)	Red Clearance Interval (sec.)				
32.2	1.0	1.4	2.1	2.8	3.5

NOTE: Based on turning path speed of 20 mph (32.2 km/h), 1.0 sec. minimum, an entry delay of 1.0 sec., and an average vehicle length of 20 ft. (6.1 m).

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