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Guidelines for Determining Traffic Signal Change and Clearance Intervals

A Recommended Practice of the Institute of Transportation Engineers

ite

A Community of Transportation Professionals

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An ITE Recommended Practice

The Institute of Transportation Engineers (ITE), a community of transportation professionals, is one of the largest and fastest-growing multimodal individual member professional transportation organizations in the world. ITE members are traffic engineers, transportation planners, and other professionals who are responsible for meeting society's needs for safe and efficient surface transportation through planning, designing, implementing, operating, managing, and maintaining surface transportation systems worldwide.

Founded in 1930, ITE serves as a source for expertise, knowledge, and ideas through meetings, seminars, and publications, and through a network of more than 16,000 members working in nearly 80 countries. Institute members serve in key positions at all levels of government, including the U.S. Department of Transportation; state, county, and local transportation agencies; metropolitan planning organizations; transit, parking, and toll authorities; as well as being employed by consulting firms and universities.

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) prepared 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 recommended practice of ITE.

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 developed in collaboration with a technical advisory committee of individuals involved in the development and review of the report. The members of the technical advisory committee are as follows:

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Peer review of this document as an ITE recommended practice was provided by a recommended practice review panel consisting of individuals with active interest and knowledge in traffic signal timing. The following individuals were members of the panel:

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Following public comment and additional development through the technical advisory committee and peer review panel, a notice of intent to adopt the recommended practice was issued to interested parties and the contents of the recommended practice were appealed twice. An appeals panel was convened and provided direction to ITE modifying the recommended practice in response to the appeals received. The technical advisory committee and peer review panel were not reconvened to reach consensus on these changes. The ITE International Board of Direction ultimately approved the revised recommended practice.

Federal, state, and local agencies, other governmental offices, private enterprises, or other organizations employ certain individual volunteer members of the Institute of Transportation Engineers' technical report-developing bodies. Their participation does not constitute endorsement by these government agencies of ITE's report-developing bodies or any ITE guidelines developed by such bodies.

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

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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 prepared 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, peer review panel, and others who participated in the development process. 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. This report is not intended to cover specific enforcement actions to address red light running but does acknowledge that the range of values for variables used in calculating change intervals and the range of driver behavior they represent makes zero tolerance enforcement inappropriate. ITE strongly supports appropriate application of engineering methods to time traffic signals.

1.2 Purpose and Intended Use

ITE's intent is for the 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, maintaining reasonable traffic flow, and providing for the movement of vehicles and pedestrians. The goal of the 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. In this report, ITE indicates where there has been sufficient theoretical work, research, and practice information to reach a consensus recommendation. This is not true for all potential elements or aspects of the process to determine yellow change and red clearance intervals, and ITE recommends areas for further research as a result. Individual agencies may choose to extend their policies beyond the provisions in this recommended practice with appropriate engineering methods, procedures, documentation, and application of engineering judgment.

This recommended practice was written primarily for an audience of engineers engaged in the activity of determining yellow change and red clearance intervals. The user of this recommended practice is strongly encouraged to read the contents in their entirety. It is recognized that proper application of these intervals is dependent upon correct use of field equipment and engineering design applications. The engineer should necessarily 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
- 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 are 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, webinars, and public comment 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
- Notice of availability posted to ITE Community All Member Forum, February 25, 2015 and subsequent conversation in the forum followed
- Webinar held on the Proposed Recommended Practice on March 25, 2015
- Public comment period on Proposed Recommended Practice from February 25, 2015 through July 31, 2015
- Notice of Intent to Adopt by the ITE International Board of Direction appeals period open from September 14, 2018 through October 15, 2018
- Second Notice of Intent to Adopt by the ITE International Board of Direction appeals period open from May 8, 2019 through June 10, 2019
- Meeting of Appeals Panel held August 28, 2019

Comments or direction resulting from these sessions have been reviewed and 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 the following:

- 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*.[6] 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 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 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.

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 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.[7] Given the intent of the yellow indication is to be a warning, drivers may enter the intersection until the red signal indication is displayed.[a] 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 reasonable duration of yellow change and red clearance intervals that does not compromise intersection safety while retaining 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 the following multiple methods for determining traffic signal yellow change and red clearance intervals:[b]

- 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
- Extended kinematic equation model
- Conflict zone method
- Rational models method

[a] Note that in restrictive law states vehicles also cannot be in the intersection with a red signal indication.
[b] Please note that equations in this section are provided in the units as noted in the original reference.

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)[c] 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:

$$v_f^2 = v_o^2 + 2ad \qquad (1)$$

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:

$$0 = v_o^2 + 2ad \qquad (2)$$

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

$$d = \frac{v_o^2}{2a} \qquad (3)$$

In the foundational paper by Gazis, Herman, and Maradudin[8] for the condition of a driver coming to a complete stop before entering the intersection, the authors define the critical distance (shown in Figure 2.1 and Figure 2.2) as:

$$x_c = v_o\delta_2 + \frac{v_o^2}{2a_2^*} \qquad (4)$$

Where:

x_c	=	critical distance (sec.);
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.

[c] The term is commonly referred to in literature as "perception-reaction time." This time interval consists of the time for the driver to perceive the amber clearance signal, while the reaction time of the driver-vehicle system is composed of the time for the drive to depress the brake pedal and the vehicle to react to this input by applying the brakes.

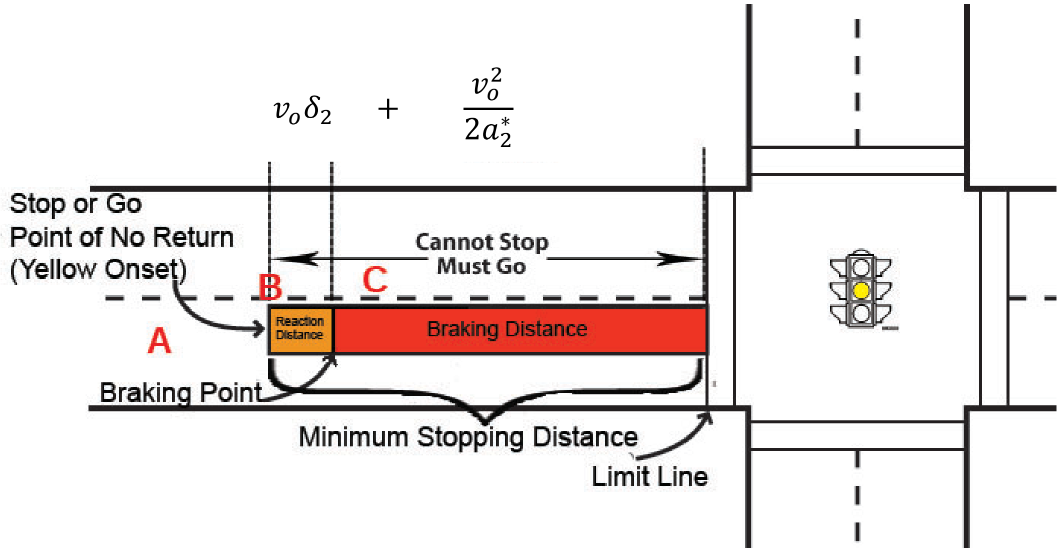

Figure 2.1: Intersection Diagram for Gazis, Herman, and Maradudin Derivation

Source: Adapted from Beeber, J., "Yellow Change Intervals for Turning Movements Using Basic Kinematic Principles," unpublished paper submitted to ITE, August 21, 2019, pg. 2.

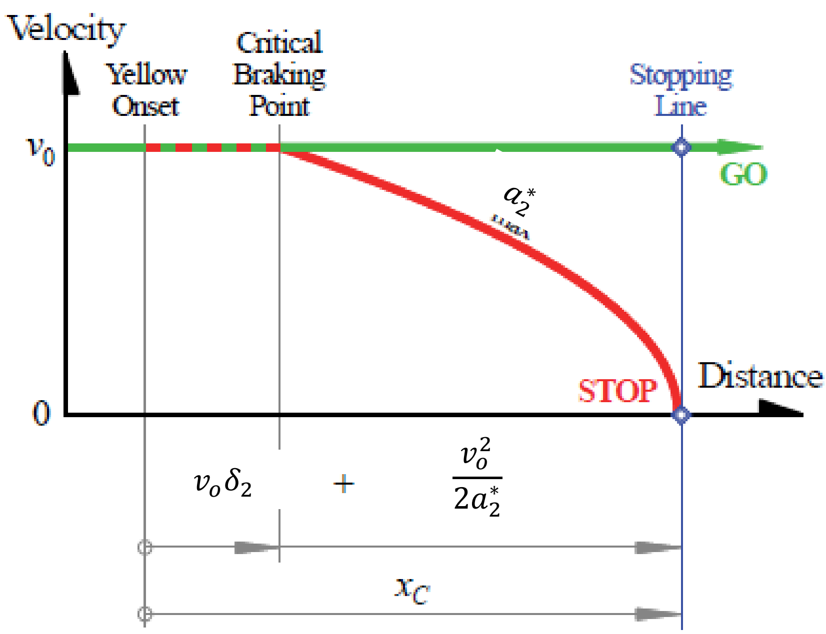

Figure 2.2: Velocity vs. Distance in Gazis, Herman, and Maradudin Derivation

Source: Adapted from Järlström, M., "An Extended Yellow Change Interval Solution Derived from GHM's Critical Distance, Revision A," unpublished paper submitted to ITE, February 1, 2019, pg. 1.

By noting that when x_o (the maximum distance the car can be from the intersection at the start of yellow and still clear without accelerating) corresponds to x_c , a minimum yellow indication duration is calculated for the change and clearance time of an approaching vehicle by dividing the critical distance x_c by the original velocity v_o:

$$\tau_{min} = \delta_2 + \frac{\frac{1}{2}v_0}{a_2^*} + \frac{(w+L)}{v_o} \qquad (5)$$

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 not a subject of their paper, they state…,

> *"Analogous results may be obtained for two closely space traffic lights, as in the case of crossing a divided highway. However, this case is rather complicated and will not be discussed here. There are other variations to the problem of the dilemma zone such as the case of a vehicle approaching an intersection at slow speed with the intention of making a turn. This is a case of known practical difficulty and some information can be obtained from the present analysis with w taken equal to the distance traversed while turning."*

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*[9] provides a comprehensive review of these models and formative research prior to this 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[10] provides Equation 6 and Equation 7 for calculating the yellow change and red clearance interval, or change period:

$$Y = t + \frac{v}{2a+2Gg} \qquad (6)$$

$$R = \frac{W+L}{v} \qquad (7)$$

Where:

Y = yellow change interval (sec.);
t = perception-reaction time (typically 1 sec.);
v = design speed (ft./sec.);
a = deceleration (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.);
L = length of vehicle (typically 20 ft.).

The earlier ITE *Traffic Engineering Handbook*, 4th Edition[11] and *Determining Vehicle Change Intervals: A Proposed Recommended Practice*[12] 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.

$$r = \frac{w+L}{v} \quad (8)$$

$$r = \frac{P}{v} \quad (9)$$

$$r = \frac{P+L}{v} \quad (10)$$

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,[13] 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)[14] 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*[15] published in 2008, also suggests applying the kinematic equation method. The subsequent *NCRHP Report 812: Signal Timing Manual, Second Edition*[16] published by the Transportation Research Board takes the same approach referencing ITE's material and *NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections* [17].

NCHRP Project No. 03-95, and the accompanying *NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections* [18] reviews the background, literature, and collected new data for intersection parameters at 83 signalized intersection approaches 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. Additionally, the authors of the NCHRP report formulated the red clearance interval with a 1 second 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:

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

$$R = \frac{W+L}{1.47V} - 1 \quad (12)$$

Where:

Y = yellow change interval (sec.);
t = perception-reaction time (set at 1.0 sec.);
V = 85th percentile approach speed (mph);
a = deceleration (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.);
L = length of vehicle (set at 20 ft.).

Unfortunately, there is some confusion in the literature about the term "dilemma zone." Some authors have used this term as Gazis did: referring to a portion of the intersection approach where a driver can neither comfortably stop nor continue on to the intersection before the end of the yellow change interval. A properly timed yellow change interval eliminates this dilemma zone by providing reasonable drivers with the ability to either stop or proceed based on what is physically possible.

The NCHRP report notes that an "indecision zone" was identified in research,[19, 20, 21, 22] and referred to as a "type II dilemma zone" referring to the portion of the intersection approach where individual driver's response may be different with respect to the onset of the Yellow Change interval depending on how timid or aggressive they may be. This zone has been defined as a distance interval with a driver stopping probability between 10 and 90 percent and is appropriately related to those concepts. An "indecision zone" will continue to exist at the onset of every yellow indication, regardless of change interval durations. This is due to the fact that drivers will react differently when facing a yellow signal indication, regardless of whether adequate yellow time is provided, based on prevailing conditions. More recently, this portion of the intersection approach is more appropriately referred to as the "indecision zone" rather than the "dilemma zone"

The ITE *Traffic Control Devices Handbook*, 2nd Edition[23] provides two forms of the equations for the yellow change and red clearance intervals based on the kinematic equation method 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

In addition to the kinematic 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*,[24] as a technique used by practitioners for determining yellow change intervals. This report does not promote the rule-of-thumb method as a recommended practice; it simply presents it as an approach used by some public agencies, in the absence of data or resources necessary to develop a change interval based on intersection geometry and traffic flow characteristics, for setting the duration of the yellow signal. 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.,[25] Frantzekakis,[26] and Wortman et al.[27] 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[28] 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).

$$\tau_{min} = \frac{A+W+L}{V_o} \qquad (14)$$

Where:

- τ_{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[29] 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 observations of the driver and acceleration of the conflicting vehicle (see Equation 15).

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

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 Protected Left-Turn Movements

Liu, Yu, Saksit, and Oey[30] modified the kinematic model proposed by Gazis et al. to accommodate the entering speed of a left-turning vehicle under a protected signal phase. 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 protected left-turn movements is shown in Equation 16.

$$y_t = 2\,\frac{\left(\delta_{-} + \frac{v_l}{2a_{-}}\right)}{\left(1+\frac{v_i}{v_l}\right)} \qquad (16)$$

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 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.

Yu, Qiao, Zhang, Tian, and Chaudhary[31] 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.

$$y_t = 2\,\frac{\left(\delta_{-} + \frac{v_l}{2a_{-}}\right)}{\left(1+\frac{v_i}{v_l}\right)} + T_{vi} \qquad (17)$$

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 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.[32]

In addition to proposing a yellow change interval model for left-turn movements, Liu, Yu, Saksit, and Oey also modified the kinematic equation 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);
- $\tilde{w}_t$ = $w_t + L$
- w_t = straight line distance from the stop line to the extension of the midpoint of the departure lane (m);
- L = length of vehicle (m);
- w_l = straight line distance from the extension of the midpoint of the approach lane to the clearance line (m);
- Φ = intersecting angle between vehicle approach and departure direction (radians); 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.

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

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 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 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.

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

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.

Extended Kinematic Equation Model

Gazis, Herman, and Maradudin noted other variations to the dilemma zone problem including, “the case of a vehicle approaching an intersection at slow speed with the intention of making a turn.” Järlström[33] proposed an extended solution to kinematic equation method for the case of a vehicle approaching an intersection at a high(er) speed then slowing with the intention of making a turn. This formulation addresses the variation cited by Gazis, Herman, and Maradudin as well. This extended kinematic equation model approach adds a new intersection entry velocity variable, v_1 , to allow for deceleration within the critical distance. The resulting Equation 22 is analogous to Equation 4 but broken into three parts instead of two for a vehicle to slow at a maximum uniform safe and comfortable deceleration to the intersection stop line (shown in Figure 2.3):

$$x_c = v_0 t_{PR} + \frac{v_0^2 - v_1^2}{2a_{max}} + \frac{v_1^2}{2a_{max}} \quad (22)$$

$$v_0 \geq v_1 \geq 0$$

Where:

x_c = Critical distance (ft. or m)
t_{PR} = Maximum allocated driver-vehicle perception-reaction time (sec.)
v_0 = Maximum uniform initial/approach velocity (ft./sec. or m/sec.)
v_1 = Maximum uniform intermediate/entry velocity (ft./sec. or m/sec.)
a_{max} = Maximum uniform safe and comfortable deceleration (ft./sec./sec. or m/sec./sec.)

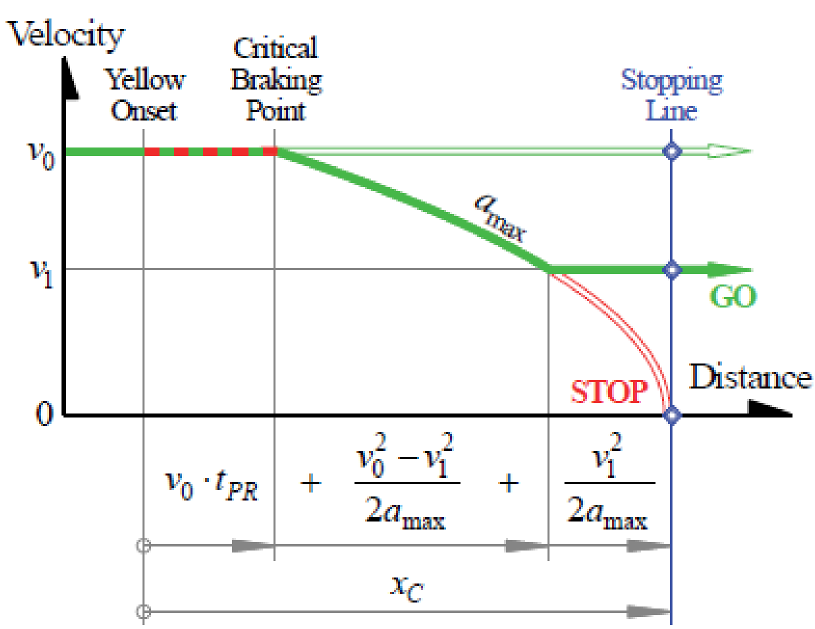

Figure 2.3: Velocity vs. Distance Graph of Järlström Extended Derivation

Source: Järlström, M., “An Extended Yellow Change Interval Solution Derived from GHM’s Critical Distance, Revision A,” unpublished paper submitted to ITE, February 1, 2019, pg. 3.

In a similar manner to Equation 5, this equation can be converted and reduced to provide an extended minimum yellow change interval duration of an approaching vehicle for a movement where a vehicle is decelerating within the critical distance.

$$Y_{EP} \geq t_{PR} + \frac{v_0}{a_{max}} - \frac{v_1}{2a_{max}} \quad (23)$$

Where:

Y_{EP} = Extended permissive yellow signal duration (sec.)
t_{PR} = Maximum allocated driver-vehicle perception-reaction time (sec.)
v_0 = Maximum uniform initial/approach velocity (ft./sec. or m/sec.)
v_1 = Maximum uniform intermediate/entry velocity (ft./sec. or m/sec.)
a_{max} = Maximum uniform safe and comfortable deceleration (ft./sec./sec. or m/sec./sec.)

When $v_0 = v_1$ the result of the extend model is the same as the Gazis, Herman, and Maradudin kinematic equation method. Järlström further notes a case when $v_1 = 0$ results in the minimum time to traverse the braking distance to a stop.

Järlström's work was supported by Beeber[34] in supplemental material by positing several decision scenarios regarding the location where drivers can decelerate relative to the critical distance approaching a signalized intersection. These include the following:

- Before reaching the minimum stopping distance, or critical distance (Figure 2.4 and Figure 2.5);
- Within the perception-reaction distance (Figure 2.6 and Figure 2.7); and
- Within the braking distance (Figure 2.8 and Figure 2.9)

The boundary condition for the three scenarios is shown in Figure 2.6, which results in the longest time to traverse the braking distance, irrespective of specific values of initial speed and intersection entry speed. This model provides the longest time for a vehicle to the critical distance. This duration is also the minimum time for a vehicle too close to the intersection when the yellow interval is displayed to travel the distance to the stop line before the signal changes to a red indication.

$$Y_{min} = t_{PR} + \frac{v_0 - v_E}{a} + \frac{v_E}{2a} \quad (24)$$

Where:

Y_{min} = minimum yellow change interval (sec.)
t_{PR} = perception-reaction time (sec.)
v_0 = initial approach velocity (ft./sec.)
v_E = Movement entry velocity (ft./sec.)
a = Maximum deceleration (ft./sec./sec.)
$v_E = v_0$ The protocol yields the formula for the kinematic equation
$v_E = 0$ The protocol yields the formula for the time for a vehicle to come to a complete stop

Assuming:

- The vehicle is traveling in free-flow conditions (unimpeded, not within a queue, etc.).
- The yellow indication illuminates at the moment the vehicle arrives at the minimum stopping distance.
- When the yellow indication illuminates, the vehicle's approach velocity is no greater than the 85th percentile speed or the posted speed limit, whichever is higher.

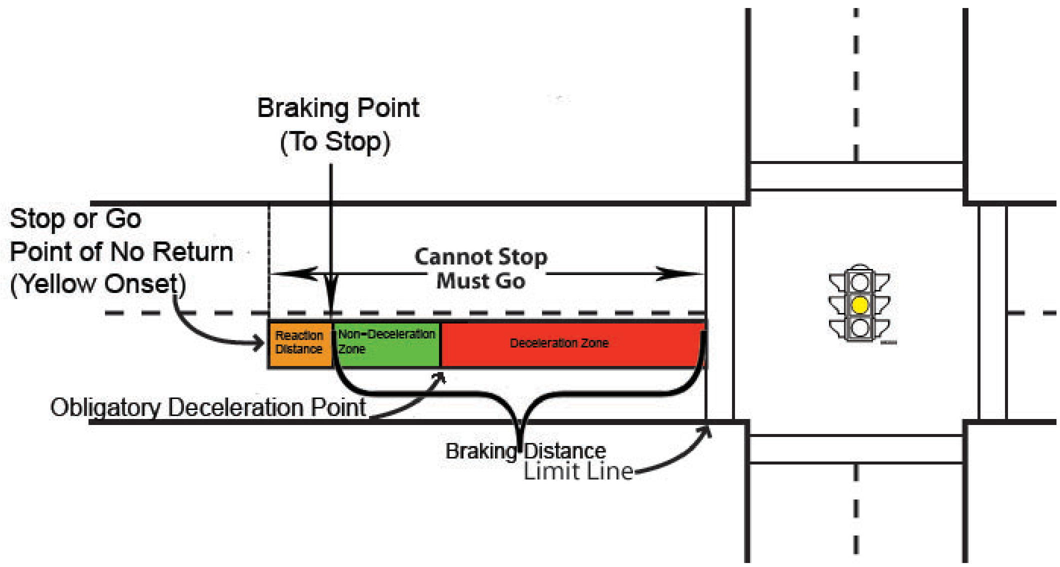

Figure 2.4: Intersection Diagram for Decision to Delay Deceleration Until Reaching the Obligatory Deceleration Point

Source: Beeber, J., "Yellow Change Intervals for Turning Movements Using Basic Kinematic Principles," unpublished paper submitted to ITE, August 21, 2019, pg. 6.

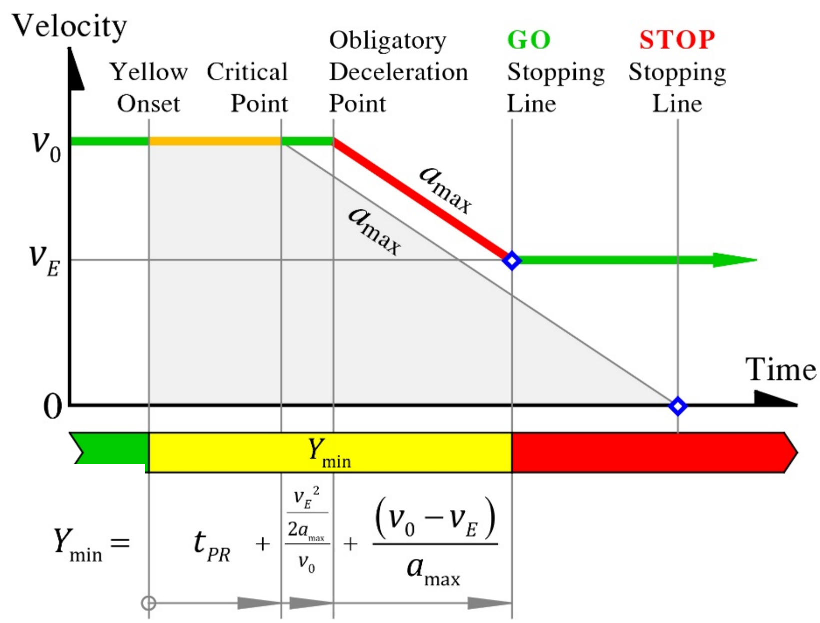

Figure 2.5: Velocity vs. Time Graph for Decision to Delay Deceleration Until Reaching the Obligatory Deceleration Point

Source: Beeber, J., "Yellow Change Intervals for Turning Movements Using Basic Kinematic Principles," unpublished paper submitted to ITE, August 21, 2019, pg. 8 (figure by Järlström, M.).

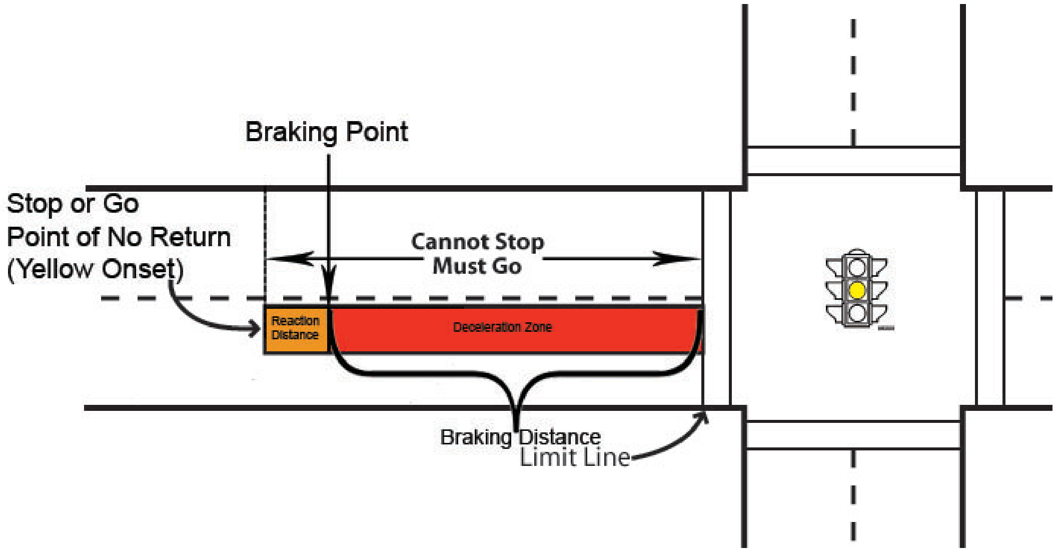

Figure 2.6: Intersection Diagram for Decision to Decelerate with Uniform (Constant) Deceleration Across the Braking Distance.

Source: Beeber, J., "Yellow Change Intervals for Turning Movements Using Basic Kinematic Principles," unpublished paper submitted to ITE, August 21, 2019, pg. 8.

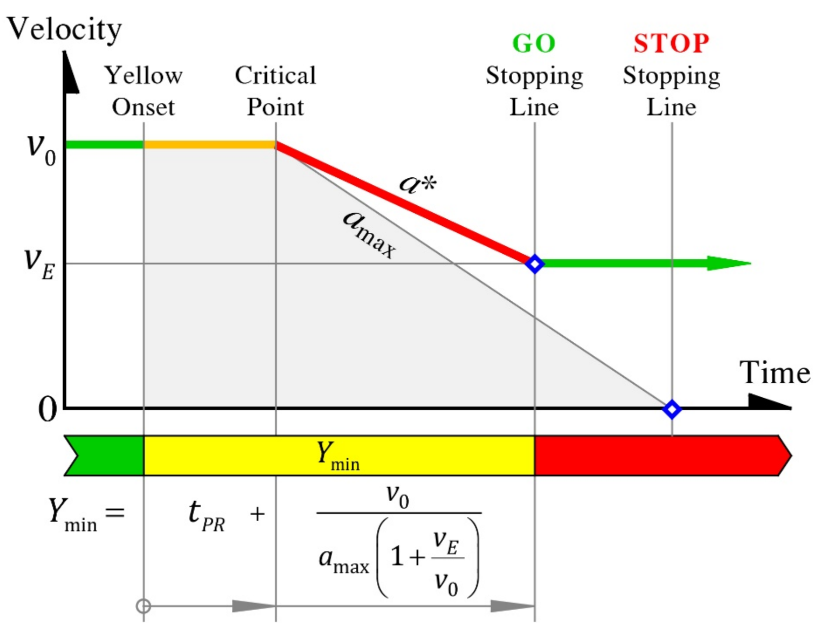

Figure 2.7: Velocity vs. Time Graph for Decision to Decelerate with Uniform (Constant) Deceleration Across the Braking Distance.

Source: Beeber, J., "Yellow Change Intervals for Turning Movements Using Basic Kinematic Principles," unpublished paper submitted to ITE, August 21, 2019, pg. 10 (figure by Järlström, M.).

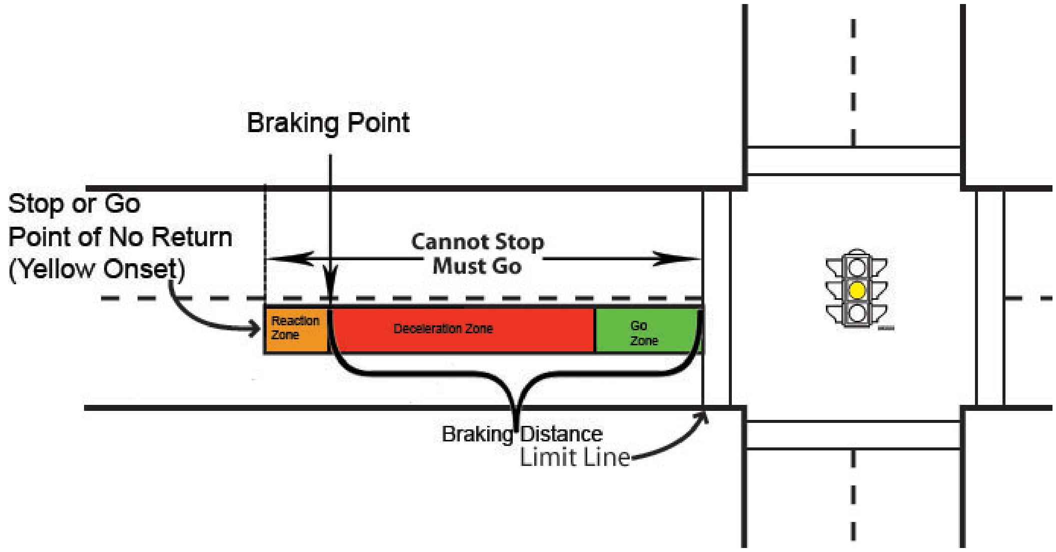

Figure 2.8: Intersection Diagram for Decision to Decelerate with Maximum Deceleration Beginning at the Braking Distance to a Target Entry Velocity and Continue at that Velocity into Intersection

Source: Beeber, J., "Yellow Change Intervals for Turning Movements Using Basic Kinematic Principles," unpublished paper submitted to ITE, August 21, 2019, pg. 10.

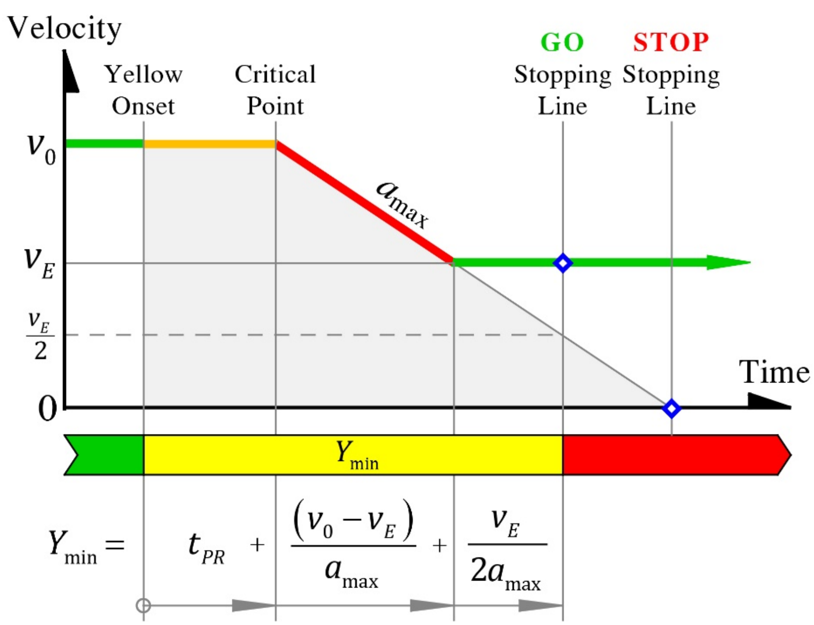

Figure 2.9: Velocity vs. Time Graph for Decision to Decelerate with Maximum Deceleration Beginning at the Braking Distance to a Target Entry Velocity and Continue at that Velocity into Intersection

Source: Beeber, J., "Yellow Change Intervals for Turning Movements Using Basic Kinematic Principles," unpublished paper submitted to ITE, August 21, 2019, pg. 12 (figure by Järlström, M.).

Conflict Zone Method

Muller, Dijker, and Furth[35] 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.

$$R = t_{exit} - t_{entrance} \tag{25}$$

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.);
$t_{entrance}$	=	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).

$$t_{exit} = \frac{s_{exit}}{v_{exit}} \tag{26}$$

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.

$$t_{entrance} = \begin{cases} t_r + \sqrt{\dfrac{2 \times s_{entrance}}{a_{acc} - a_{dec}}}, s_{entrance} \leq s_{critical} \\ t_r + \dfrac{s_{entrance}}{v_{max}} + \dfrac{v_{max}}{2 \times (a_{acc} - a_{dec})}, s_{entrance} > s_{critical} \end{cases} \tag{27}$$

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. The general conflict zone methodology was validated by Li, et al.[36]

Rational Models Method

Fitch, Shafizadeh, Zhao, and Crowl[37] 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.

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.

$$R = t_c - t_{min} \tag{28}$$

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:

$$t_{min} = \sqrt{\frac{2D}{a_s - a_r}} \tag{29}$$

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 of 10 (ft./sec./sec.) and an acceleration of 15 (ft./sec./sec.), as suggested by the researchers, Equation27 becomes

$$t_{min} = 0.283\sqrt{D} \tag{30}$$

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. Respondents indicated they knew 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, respondents indicated they knew 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. 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 other formulae, cited policies, or methods used in practice. Information regarding these other methods is summarized in Table 2.2. It should be noted that a number of the other policies cited by agencies are kinematic equation based.

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	Percent
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 value by approach speed applied to all intersections	38	18
Other	40	18
TOTAL	217	100

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

Other Formula	Other Cited Policy	Other Methods
3.5 to 4 sec. on low speed approaches and 5+ sec. 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-published information
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 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-published information for change intervals	Kinematic equation compared with uniform values

Note: Table 2.2 agency survey responses were submitted prior to the 2014 update of the California MUTCD. Columns are not referential to each other within a row.

Recommendation

As part of the development process, relevant research on different methods of calculating traffic signal change intervals was reviewed and suggestions for alternative methods received 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 certain geographic areas, sufficient evidence does not exist to support that these methods are improvements over methods based on the kinematic equation. Additionally, specific research using the kinematic equation and extensions thereof has been completed regarding protected left-turn phases that provides support applicable to that particular type of signal indication. Based on an evaluation of the available information, the approach based on the extended kinematic equation model is preferred for determining the yellow change and red clearance intervals for through and turning movements as currently formulated in the most recent information 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. These versions of the kinematic equation method are 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-based 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 specifically in the context of turning movements, 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 extended kinematic equation model expands on the commonly used form of the kinematic equation to address the issue of those turning vehicles which enter the intersection at an entry speed less than the 85th percentile approach speed. When applied to through movements where vehicles enter the intersection at the 85th percentile approach speed, the extended kinematic equation model reduces to the common form of the kinematic equation. 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 and speed with which a reasonable drive enters the intersection.

The extended kinematic equation model with the addition of a grade adjustment factor (as recommended in Section 2.10) is the basis for the rest of the discussions in this recommended practice. The equations are noted below in U.S. units:

$$Y \geq t + \frac{1.47(V - V_E)}{a + 32.2g} + \frac{1.47V_E}{2a + 64.4g} \qquad \text{(U.S. units)} \quad (31)$$

$$R = \left[\frac{W + L}{1.47V_E}\right] - t_s \qquad \text{(U.S. units)} \quad (32)$$

Where:

Y = minimum yellow change interval (sec.);
t = perception-reaction time (sec.);
V = 85th percentile approach speed (mph);
V_E = intersection entry speed (mph);
a = deceleration (ft./sec./sec.);

g = grade of approach (percent/100, downhill is negative grade);
R = red clearance interval (sec.);
W = distance to traverse the intersection(width), stop line to far side no-conflict point along the vehicle path (ft.);
L = length of vehicle (ft.);
t_s = conflicting vehicular movement start up delay (sec.).

The equations are noted below in metric units:

$$Y \geq t + \frac{0.28(V - V_E)}{a + 9.8g} + \frac{0.28V_E}{2a + 19.6g} \qquad \text{(Metric units)} \quad (33)$$

$$R = \left[\frac{W + L}{0.28V_E}\right] - t_s \qquad \text{(Metric units)} \quad (34)$$

Where:

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

This formulation of the kinematic approach comes from the combination of most recent research document, NCHRP Report 731: *Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections*,[38] and from materials (on derivation of the extended kinematic equation model) submitted to ITE and deliberations of advisory members of the community of practice. In this formulation, distinct from the more general kinematic equation method, consideration is given to 1) the intersection entry speed for turning vehicles, 2) approach grade, and 3) 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 1.0 second. Additionally, the equations provided in the formulation were converted to allow input to the equations 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[39] 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)[40] 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 and territories 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 prohibit vehicles from crossing or being in the intersection on red. The statutes in these four states conflict with the MUTCD.

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 that 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[41]summarizes the history of the PRT variable. The report states a 1.1 sec. PRT was used implicitly in the 1941 and 1950 editions of the ITE *Traffic Engineering Handbook*, 2nd Edition.[42]

The 1960 Gazis et al.[43] 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*.[44] All subsequent editions of the *Traffic Engineering Handbook* have suggested this 1 sec. PRT.

In 1977, Williams[45] 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[46] 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 [47] references Tarawneh's 1991 thesis from the University of Nebraska,[48] which supported a 1.5 sec. PRT. The work reviewed the history of PRT and examined human factors affecting older-driver PRT.

A 1995 experimental study by Knoblauch et al.,[49] 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[50] 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[51] 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[52] 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.

Mezaki, et al.[53] observed data from vehicle recorders placed in five different taxi cabs operating within a major urban area in Japan. The recorders included a video capture component, used to observe the onset of yellow and red intervals, along with a device to measure the application of vehicle braking. Data was collected over a 5month period. Overall, a mean PRT of 0.75 sec. was observed from the drivers, although a significant number of PRT values from the collected data were negative, indicating that the drivers applied the brake prior to the onset of yellow (perhaps anticipating the end of the green interval).

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.[54]

Research was performed as part of NCHRP Report 731[55] and further documented by Gates et al.[56]at 83 signalized intersection approaches in five states based on an initial data set of 7,482 vehicle reports. Brake-response time, defined as the difference in time between the observed start of the yellow indication and vehicle brake lights, was measured for more than 2,400 vehicles in the decision zone that were first to stop after the yellow onset. The research found the following:

- 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 increased (i.e., drivers decelerating more rapidly used longer PRT times)
- 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.[57] documentation of the data analysis for the NCHRP report, he notes that

> *"…PRT and deceleration should be jointly considered as motorists do not select these variables independently of each other. Drivers do not select these variables independently of one another. 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[58]published a discussion of Wortman and Matthias'[59] 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.

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.

Table 2.3: Perception-Reaction Times Used in Practice

Perception Reaction Time (sec.)	No. of Responses	Percent
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**

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.

As shown by a number of cited studies, there is an important difference between perception-reaction times that were actually observed vs. perception-reaction times that were possible if drivers needed to react more quickly. In other words, a driver that is 220 feet away from the intersection at 30 miles per hour from the intersection when the light turns yellow does not need to react as quickly as a driver that is 130 feet away at the same speed. As a result, observed perception-reaction times are often biased toward higher values than possible perception-reaction times. This is borne out by the data contained in a study by Gates, Noyce, Laracuente, and Nordheim[60] which found that 85% of drivers could react within 1.0 second if they needed to. Additionally, Gates, McGee, Moriarty, and Maria[61] documented the correlation between PRT and deceleration rates such that longer PRT will result in higher deceleration rates, with the net impact to the application of the kinematic equation-based methods to a change in PRT limited due to the interplay of the two parameters.

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*[62] 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* [63] 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;[64] 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.[65] 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*[66] 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 37.

$$EV85 = 7.675 + 0.98xPSL \tag{35}$$

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[67] 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 among all 48 sites in the study was 9.5 mph (15.3 km/h). The FHWA memorandum[68] 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[69] examined approach speeds of 3,632 through movement vehicles at 83 signalized intersection approaches in five states. 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. 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 of the respondents used posted speed limits, compared with 18 percent who used 85th percentile approach speeds.

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 or some measure of speed 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.

Table 2.4: Approach Speed Used in Practice

Speed	No. of Responses	Percent
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 Response Descriptions of Speed Measures for Change Interval	
85th percentile where available	Posted speed for new, operating speed for existing
85th percentile for yellow, posted speed for red	Posted speed, unless 85th percentile speed is known
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 limit established based on the 85th percentile speed
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	We estimate speed from speed limit and familiarity
Estimated 15th percentile and 85th percentile speeds, whichever yields longer time	Posted, unless known 85th percentile higher (higher # used)
For yellow change and red clearance intervals, we "follow" ITE recs (engineering judgment used)	Recommended as 85th percentile or 15th percentile speed
Greater of speed limit or 85th percentile (if known)	High range of comfortable speed after trial runs
Mostly 85th percentile, but use posted speed in special situations	Speed limit with consideration to 85th percentile speed
Posted + 5 mph (8 km/h)	Through = posted speed
Posted speed for throughs or rights and lower speed for lefts	Usually posted speed limit or 85th percentile when handy
Posted + 5 mph (8 km/h) for amber, posted for red clearance interval	Varies by location (85th percentile or Posted speed limit)
Yellow V = Posted + 5 mph (8 km/h); Red Clearance Interval V = Posted	We use posted, but considering using 85th percentile.
Posted or prima facie speed	We use the greater of 85th percentile or posted
Posted speed and observation	Yellow interval, posted speed or prima facie speed
California MUTCD (2014, Rev 1)[70] or V = posted speed + 10 (speed limit <= 25 mph)	V= posted speed + 7 (speed limit >= 30mph)

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	Percent
Posted Speed Limit	79	52
85th Percentile Approach Speed	27	18
Design Speed	2	1
Other	43	29
TOTAL	**151**	**100**

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 Response Descriptions of Speed Measures for Clearance Interval		
(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 through, 15 mph (24 km/h) for left turns
10th percentile for red clearance		Posted speed for new, operating speed for existing
50th percentile approach speed		Posted unless engineering judgment dictates other
50th percentile speed		Posted unless speed evaluation is available
Same as above but -1 sec. for left turns only		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		85th percentile but time generally not to exceed 2 sec.
85th percentile approach speed and speed limit		Same speed
Default value is 1.0 sec. and 2.0 sec. if needed		Posted speed limit
Engineering judgment		Typically, we use a 2.0 sec. all-red interval
Engineering judgment on various factors		Uniform 1.0 sec. unless accident problems persist
Field observations		Uniform setting of 2-sec.
Follows the same as yellow change		Time from limit line to last point of collision at 10th percentile speed
For red clearance interval, we "follow" ITE recs (engineering judgment used)		Usually posted speed limit or 85th percentile when handy
Mostly 85th percentile, but use posted speed in special situations		Varies: Some use fixed values / incorporated speed
Greater of speed limit or 85th percentile (if known)		For left turns, turn execution speed

Recommendation

The preferred method for representing approach speed is to use the measured 85th percentile approach speed for the yellow change interval and red clearance interval calculation as recommended by available research such as NCHRP Report 7314. 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 has not been conducted, the 85th percentile approach speed for through movements may be estimated and substituted for V85 by the following equation for calculating the yellow change interval in U.S. units:

$$V_{85E}\text{ (through)} = SL + 7 \qquad \text{(U.S. units)} \quad (36)$$

Where:

V_{85E} = estimated 85th percentile speed, in mph; and

SL = posted speed limit, in mph.

Or in metric units:

$$V_{85E}\text{ (through)} = SL + 11 \qquad \text{(Metric units)} \quad (37)$$

Where:

V_{85E} = estimated 85th percentile speed, in km/h; and

SL = posted speed limit (km/h).

The relationship between 85th percentile speed and the posted speed limit is based on the recommended guidelines created from results of field observational measurement at 83 signalized intersection approaches as documented in NCHRP Report 731[71] 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. This value should not be less than the posted speed limit. 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, observed local conditions, 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. As with the speed used for the yellow change interval, this speed should not be less than the speed limit for the red clearance interval. 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. 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 by age and gender were also found.

As noted in *A History of the Yellow and All-Red Intervals for Traffic Signals,*[72] the deceleration of an approaching vehicle has the greatest effect on the variance of the calculated change interval.[73] 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 as constants within the equation or as a variable of the stopping distance. Gazis et al.[74] concluded a 10 ft./sec./sec. (3 m/sec./sec.) deceleration was appropriate based on an observational study of 87 drivers. The 1965 edition of the *Traffic Engineering Handbook*,[75] however, suggested 15 ft./sec./sec. (4.6 m/sec./sec.) as a reasonable deceleration. *A History of the Yellow and All-Red Intervals for Traffic Signals*[76] cites Parsonson and Santiago[77] for suggesting that the source of 15 ft./sec./sec. (4.6 m/sec./sec.) deceleration 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*[78] was modified and suggested applying a 10 ft./sec./sec. (3 m/sec./ sec.) deceleration, which was supported by Gazis et al.[79] 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[80] reported mean deceleration 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 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[81] 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 was 9.5 ft./sec./sec. (2.9 m/sec./sec.).

Referenced in the *Handbook for Designing Roadways for the Aging Population*,[82] Knoblauch et al.[83] observed higher deceleration in an experimental study. Mean deceleration 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[84] also examined deceleration for 77 drivers approaching an intersection at 70 km/h (43.5 mph). Findings supported a significant relationship between deceleration and time to stop line and age. Deceleration decreased as drivers were farther from the stop line. For a range of controlled time to stop line values, mean deceleration ranged from 8.2 to 18.0 ft./sec./sec. (2.5 to 5.5 m/sec./sec.). Results indicated deceleration were slower for older drivers. The mean deceleration 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[85] 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 and time to the stop line, driver age, and driver gender. Similar to Caird et al.'s results, drivers had slower deceleration 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 compared to middle-aged (age 40 to 59) drivers. The mean deceleration 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[86] indicated a strong relationship between deceleration and approach speed. For approach speeds of less than 40 mph (64.4 km/h), the 50th percentile deceleration was 10.9 ft./sec./sec. (3.3 m/sec./sec.), while the 85th percentile deceleration 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 decreased to 8.3 ft./sec./sec. (2.8 m/sec./sec.), while the 85th percentile deceleration 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 for calculating the stopping sight distance.[87] No guidance is given on applying this value for calculating change intervals.

Cheng et al.[88] proposed the following more universal equation for estimating deceleration, accounting for the possibility of less drastic deceleration on sections of roadway with curves or other superelevation features:

$$a = \sqrt{g^2 f^2 - \left(\frac{\left(\frac{V}{3.6}\right)^2}{R} - ge\right)} \tag{38}$$

Where

a = the rate of deceleration
g = acceleration due to gravity
f = coefficient of pavement friction
V= vehicle speed
R = radius of curvature
e = rate of superelevation

In the case of level terrain (R = +8 and e = 0), the computed rated of deceleration will be the same as that utilized in the kinematic equation method.

NCHRP Report 731[89] examined this parameter as well and was further documented by Gates et al.[90] 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 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). PRT and deceleration 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 and proposed the use of a 10 ft./sec./sec. (3 m/sec./sec.) rate.

Current Practice

The survey respondents were asked about deceleration for the kinematic equation. The overwhelming majority reported using 10 ft./sec./sec. (3 m/sec./sec.) or a similar value (Table 2.8). Primary comments received were related to the concern that deceleration would need to be measured in the field.

Table 2.8: Values Used for Deceleration

Deceleration Rate	No. of responses	Percentage
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**

Recommendation

Guidance on applying the deceleration typically provides average values for deceleration rather than measurement of the rate in the field. Based on the available research, a uniform deceleration of 10 ft./sec./sec. (3 m/sec./sec.) is appropriate for most users. The extended kinematic equation model assumes uniform deceleration though this is an oversimplification. However, a uniform deceleration of 10 ft./sec./sec. (3 m/sec./sec.) is a mean deceleration 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[91] 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[92] 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[93] provides three equations for calculating the red clearance interval based on the presence of pedestrians (Equations 8, 9, and 10).

The 2009 MUTCD as revised[94] defines the intersection width for an intersection controlled by a traffic signal as the following:

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[95] defines an intersection as the following:

> *"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."*

And further clarifies that approaching traffic,

> *"… shall stop at a clearly marked stop line, but if none, before entering the crosswalk on the near side of the intersection, or if none, then at a point where the driver has a view of approaching traffic on the intersecting roadway prior to entering it."*

NCHRP Report 731[96] 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 line to the curb-line extension or outside edge of the farthest conflicting vehicular movement, along the vehicle's travel path. Figure 2.10 illustrates this distance. When there is frequent pedestrian traffic or the crosswalk(s) are protected by pedestrian signals, a longer distance for intersection width from the near-side stop line to include the far-side of departure crosswalk may be selected to increase safety of all users using engineering judgment.

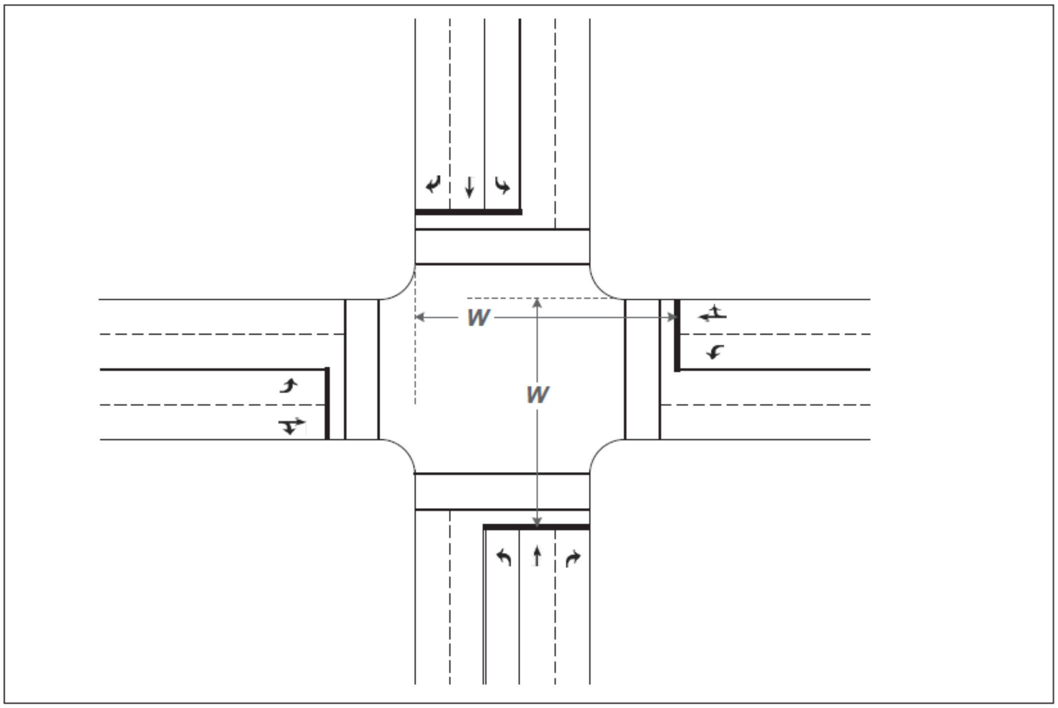

Figure 2.10: Diagram of Intersection Width for Through Movements

2.9 Vehicle Length

Literature

The vehicle length variable takes into account the length of a large majority of four-wheel vehicles that clear the intersection or conflict point. In 1977, Williams[97] 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*[98] 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*[99], which suggests 15 ft. (5.2 m). The second edition of the *Traffic Control Devices Handbook*[100] uses 20 ft. (6.1 m) for vehicle length.

The Green Book provides groupings of selected vehicles ("design vehicles") to establish highway design controls.[101] 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[102] 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.

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	Percentage
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. (13.70 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[103] references the 1982 edition of the *Manual of Traffic Signal Design*[104] 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.[105] Subsequent ITE publications have included the approach grade variable in the kinematic equation-based calculation method. Grade is included in the denominator of the second term in the kinematic equation.

The FHWA *Traffic Signal Timing Manual*[106] and the subsequent *NCRHP Report 812: Signal Timing Manual, Second Edition*[107]issued by the Transportation Research Board 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*[108] 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

Since the addition of grade to the kinematic equation-based formulae, this factor' use has been consistently supported in subsequent research. A standard way of collecting intersection approach grade does not exist and sometimes may not be immediately available, particularly for streets that were paved many years ago. In many cases the grade may be a non-zero value due to drainage considerations. 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 location upstream of the area of influence of intersection operations through the intersection and use the value in the extended kinematic equation model (shown in Equation 31 and Equation 33). Alternatively, approach grade may be taken from an as-built roadway design plan or other document that specifies design criteria. Where the grade changes over this distance or where grade information is not immediately available, engineering judgment should be used to estimate an appropriate value.

2.11 Minimum and Maximum Intervals

Literature

Section 4D.26 of the 2009 MUTCD[109] 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 1971 edition of the MUTCD changed function of the yellow interval from a change and clearance role, to solely a change role and added the red clearance interval. The 6 sec. upper limit for the yellow change interval was added at this time for higher-speed approaches.[110] The literature review conducted as part of the research for *NCHRP Report 731*[111] did not find any support for these suggested values, nor definitive citations. 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 73 percent of respondents reporting minimum yellow timing values of 3.0 sec. 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 greater than or equal to 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	—	—	12	5	—	—
2.0	—	—	9	75	—	—
2.1 to 2.9	—	—	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 (56 percent) was 1.0 sec., with two-thirds of respondents reporting minimum red clearance time values greater than or equal to 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 (50 percent) was 2.0 sec., with almost two-thirds of respondents reporting agency maximum red clearance time values less than or equal to 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 25 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

It is widely accepted by practicing professionals that excessively long change intervals can lead to increases in the number of drivers proceeding through the intersection on the yellow (and potentially red). The MUTCD[112] recommends a minimum change interval of 3.0 sec. and a maximum of 6.0 sec. The practitioners should use the ranges provided in the MUTCD guidance for yellow change intervals for through movements with the allowance for engineering judgement and/or study to address special road conditions. Given that long change intervals are associated with higher speed approaches and with a desire for appropriate flexibility within the context of the application of engineering judgment and/or study, a maximum of 7.0 sec. is recommended with respect to turning movements.

The recommended practice is to calculate the red clearance interval; however, a specific minimum or maximum value is not suggested, if used.

2.12 Rounding Calculated Intervals

Literature

NCHRP Report 731[113] 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*[114] 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.[115] (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. A number of agencies (e.g., Utah DOT[116] and others) do not include vehicle length L in the calculation of the red clearance intervals for left turn movements.

Intersection entry delay and start-up delay were examined as part of the research for *NCHRP Report 731*[117] 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.[118] 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 line was 2 sec. However, with a rolling start 12 ft. (3.7 m) from the stop line, 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. Agencies may omit the length of vehicle from the calculation for wider crossing distances and left-turns when accompanied by documented engineering judgement and supporting agency policy. The subchapter on minimum and maximum values does not recommend a specific minimum or maximum value for the red clearance interval if used. An intersection entry delay factor may be subtracted from the calculated red clearance interval;

The recommended practice is to calculate the red clearance intervals; however, a specific minimum or maximum value is not suggested, if used.

2.14 Turning 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.[119, 120, 121, 122] 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[123] 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 signalized intersections approaches in five states 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 with regard to approach speed that, "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." The study placed more emphasis on data confirming that left-turn approach speeds are lower than those through movement and did not further address differences in PRT. Gates et al.[124] did not provide disaggregation by type of movement nor did the study note whether vehicle movement through the intersection affected PRT in a significant or non-significant manner.

Measuring intersection width for left-turn movements involves measuring curved vehicle path and identifying the vehicle's speed along that path. *NCHRP Report 731*[125] 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[126] 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*[127] provided the following 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[128] 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*[129] and Koppa et al.[130] discuss the perception brake reaction time of an alerted driver from earlier research and from new primary data. *NCHRP Report 400* cites 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* 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. Left-turn drivers may be more alert to anticipate the probability of stopping after entering a left-turn lane since the left-turn green indication can be relatively short, there may be a waiting queue ahead and a call usually can be placed only when drivers reach the limit line. However, additional research and study for sight-specific conditions between turning and though movement PRT would add value to this topic, especially with the more recent availability of high-resolution driver behavior data sets describing the general driving population compared to earlier studies.

For turning movements, Järlström[131] defines the initial or approach velocity at the critical distance and an intermediate or entry velocity at the intersection stop line. This approach is used to account for vehicle deceleration across the critical distance at a maximum safe and comfortable deceleration.

Beeber[132] states that the appropriate approach speed for the minimum yellow change interval is the 85th percentile approach speed as determined under free-flow conditions as determined by a speed study and should not be less than the speed limit. He challenges the approach taken in *NCHRP Report 731* by stating that the same condition extends to turning vehicles regardless of the portion of the turning vehicles that may be traveling below the speed limit. He observes that although the length and configuration of many turning lanes may require a driver to decelerate prior to entering the lane, there are other routine lane configurations where this is not the case (e.g., lane drops and very long turn lanes). He notes in these other cases that setting the intersection approach speed lower than the speed limit can create an inadvertent dilemma.

In addition, Beeber indicates that intersection entry speeds should be determined using engineering judgement and general values of 20 mph for left-turning vehicles or 12 mph for right-turning vehicles depending on curve radius of the movement. Further, in the example below he offers ITE's curve design speed formula[133] for specific calculation where there is no superelevation:

$$v_{cds} = \sqrt{15 \times R \times f} \qquad (39)$$

Where:

v_{cds} = curve design speed (mph)
R = curve radius
F = side friction factor, for speeds 20 mph or less (f=0.28)

He also provides a formulation for cases where the point of curvature for the turning path is forward of the stop line (note that a similar case can be constructed if the point of curvature for the turning path is behind the stop line).

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 following recommendations are based on adoption of the extended kinematic equation model, with variations in application depending on turning movement type and approach speed.

The yellow clearance interval values calculated by the extended kinematic equation model are significantly influenced by the values used for approach speed and intersection entry speed. Therefore, it is recommended that actual 85th percentile approach and intersection entry speeds measured through a speed study be used. Approach speed should be measured upstream of the intersection at the critical distance calculated for through movement vehicles or immediately upstream of the opening of the turn lane, whichever is closer. Accurate speed data are particularly important where approach speeds are higher (greater than 40 mph), where turning vehicles are not required to enter a separate turn lane, or the separate turn lane is very long, and where the configuration of the intersection allows for higher intersection entry speeds. Where actual speed data are not available, general values of the speed limit should be used for approach speed and 20 mph for intersection entry speed for left turns.

Use the same values for PRT and deceleration as for through movements. 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 different than 1.0 sec. is appropriate, the application of engineering judgment with supporting documentation may be used to modify this value.

For purposes of calculating the red clearance interval, if used, 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 the intersection entry speed as measured through a speed study or a general value of 20 mph (32.2 km/h), if actual speed data are not available.

For purposes of calculating the red clearance interval, if used, a start-up delay entry value may be used. However, it should be recognized that the visibility of a clearing left-turning vehicle tends to be more in the cone of vision of those drivers in the next permitted signal phase, especially when it has started on right side from the cross street approach or from the opposing approach. In addition, when the left turn has started on the left side on the cross street approach the conflict point is moving away. Therefore, drivers can more readily see the conflict and will hesitate to enter into the intersection.

In the same manner as through movements, distance along the turning path, W, for left-turn movements is the total distance as measured along the centerline turning radius at the vehicle front axle from the stop line to the departure leg curb-line extension, or outside edge of the travel lane, of farthest conflicting vehicular movement along the vehicle's natural turning path. Figure 2.11 illustrates intersection width for a left-turn movement. 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.

The following is the recommended approach to calculating the yellow change and red clearance intervals for different turning movement cases using the extended kinematic equation model:

- Protected only left-turn movements: Calculate the yellow change and red clearance intervals and adjust for grade if necessary for each approach and implement as calculated. The intervals can be of different durations for opposing approaches or adjacent through-movement phase. As noted above, in locations where approach speeds are higher (greater than 40 mph), where turning vehicles are not required to enter a separate turn lane or the separate turn lane is very long, and where the configuration of the intersection allows for higher intersection entry speeds, particular attention should be given to selecting appropriate approach and intersection entry speeds. The maximum yellow change interval should be limited to 7.0 seconds, regardless of the value calculated.
- Permissive only left-turn movements: Calculate the yellow change and red clearance intervals for the turning movements, adjust for grade if necessary, for opposing approaches, and for through movements, and select a yellow change interval value within the range calculated for both movements using engineering judgement based on volume of left turning traffic and overall intersection design (width, sight distance, safety performance, etc.). As noted above, in locations where approach speeds are higher (greater than 40 mph), where turning vehicles are not required to enter a separate turn lane or the separate turn lane is very long, and where the configuration of the intersection allows for higher intersection entry speeds, particular attention should be given to selecting appropriate approach and intersection entry speeds. The maximum yellow change interval should be limited to 7.0 seconds regardless of the value calculated. 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.

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.

Right-turn movements: There is limited research on the complex nature of driver behavior, interactions, and theoretical formulation for right-turn maneuvers and consequently, there are elements of these factors that are not fully understood. More information is necessary before making a definitive, separate recommendation for change and clearance intervals for right-turning vehicles. Thus, the recommended approach is to calculate the through movement and left-turn movement yellow change and red clearance interval times using the approaches defined in the individual cases described above.

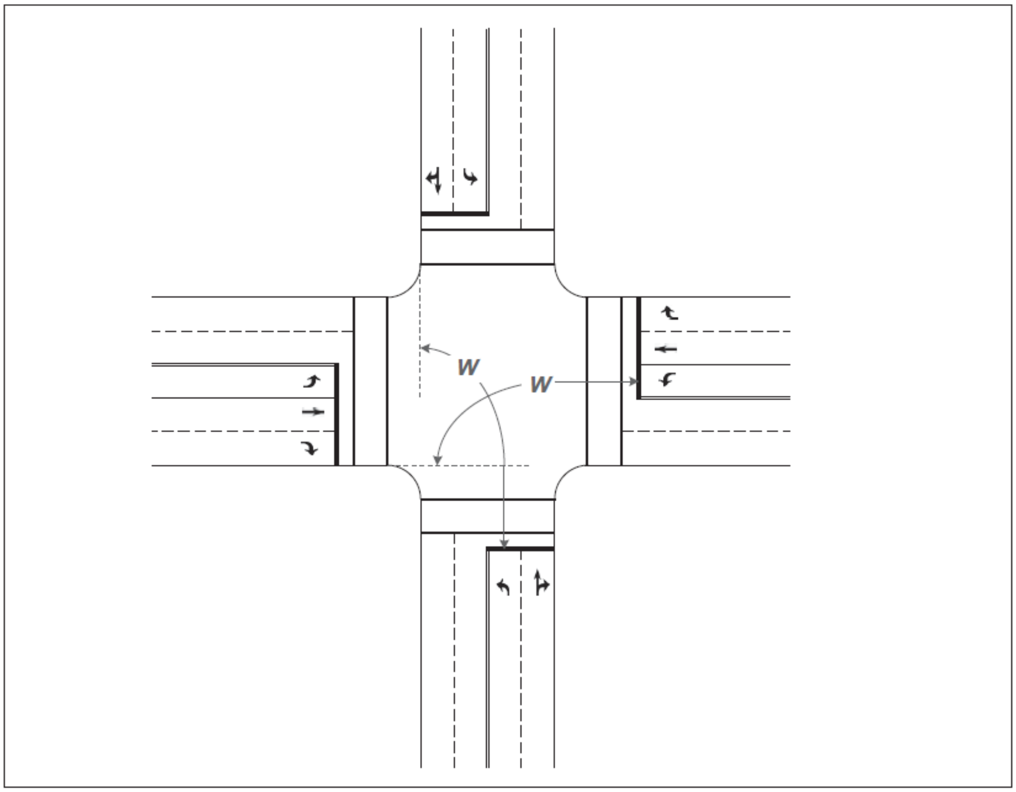

Figure 2.11: Diagram of Left-Turn Movement Path

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[134] 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 of older drivers may differ from those of the overall driving population. However, the *Handbook for Designing Roadways for the Aging Population*[135] 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).[136] This study also reported an average deceleration of 7.5 ft./sec./sec. (2.3 m/sec./sec.). The AASHTO *Guide for the Development of Bicycle Facilities*[137] 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[138] requires agencies to review and adjust signal timing on bikeways to consider the needs of bicyclists. The ITE *Traffic Control Devices Handbook*, 2nd Edition (*Handbook*)[139] 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. Section 4D.105 of the 2014 California MUTCD[140] takes a slightly different approach to providing sufficient time for bicyclists to clear an intersection. Line 14 states "For all phases, the sum of the minimum green, plus the yellow change interval, plus any red clearance interval should be sufficient to allow a bicyclist riding a bicycle 6 feet long to clear the last conflicting lane at a speed of 14.7 feet/sec plus an additional effective start-up time of 6 seconds." While this language provides sufficient guidance for a bicyclist situated at the intersection at the start of green, it does not prescribe specific intervals for the yellow change interval or red clearance interval, and thus may not allow sufficient time for bicyclists to clear an intersection who arrive late in the green phase.

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:

$$BCT_R = t + \frac{V_B}{2a} + \frac{W+L}{V_B} \qquad (40)$$

Where:

BCT_R = Bicycle crossing time—rolling entry (sec.);
t = Perception-reaction time, typically 1 sec.;
V_B = 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 BCTR from this equation may then be used to determine the bicycle clearance time:

$$BCT_R = e + Y + R \qquad (41)$$

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[141] 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[142] 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[143] 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.

Schrock and Bundy[144] 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 deceleration 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: 27 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 should necessarily 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
- 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.

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 extended kinematic equation model 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*[145] 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 kinematic equation or similar kinematic-based formulae, increasing yellow change interval duration to the formula values can significantly reduce red-light running. Studies by Bonneson and Zimmerman,[146] Harders,[147] Munro and Marshall Associates,[148] Retting et al.,[149] Van der Horst,[150] and Wortman et al.[151] 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 kinematic equation , or other formulae with a similar basis, 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 kinematic equation, or similar kinematic-based formulae, 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.[152] concluded that excessively long yellow intervals "definitely are hazardous."

Tables 5.5 and 5.6 of *NCHRP Report 705*: *Evaluation of Safety Strategies at Signalized Intersections*[153] 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-based methods, 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-based methods, 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*[154] 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*.[155]

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.[156] (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 kinematic equation, or similar kinematic-based formulae. Continued research in this area is necessary before conclusions can be drawn. The 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. At this time no published research was found 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 study by Sheffi and Mahmassani[157] compared different numbers of driver observations for estimating a "universal" dilemma zone curve, based on time remaining to reach the intersection. The study focused on high-speed, isolated intersections, and found that at approximately 4s from the intersection, the probability of a random driver stopping is 50% - the minimum time remaining before the intersection at which the driver would choose to stop is referred to as the critical time (t_{cr}). The study also determined that the significantly larger numbers of observation above 150, for the intersections studied, did not significantly change the average value of t_{cr}.

A 2005 study by FWHA asked focus group and survey participants how they would react to hypothetical traffic situations.[158] 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[159] 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[160]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[161] 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[162] 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.

- **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. Additionally, the analysis should employ some sort of approach that can identify safety benefits related to fatality and injury reduction. Further, supporting analysis incorporating non-motorized modes of pedestrian and bicycle movement would be important.
- **Impact on driver behavior and safety of yellow change intervals greater than 5.0 sec.** The current understanding in practice is very anecdotal and literature is not definitive on this issue in the context of various intersection geometries and signal phasing with the use of yellow change intervals calculated by the kinematic equation-based formulae. Continued research in this area would be important.

- **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, driver behaviors, speed limits less than 30 mph (50 km/h), turn-lane length, number of lanes, signal phasing, and conflicting bicycles and 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.
- **Perception-reaction time and deceleration 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. The more recent availability of high-resolution driver behavior data sets would add value to this type of research.
- **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 and on roadway with speed limits of 35 mph or less.** 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 and measures of effectiveness associated with approach speeds and intersection entry under yellow or red signal indications.
- **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 85th percentile intersection entry speed. 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.

- **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.
- **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.

3 - RECOMMENDED METHOD FOR DETERMINING YELLOW CHANGE AND RED CLEARANCE INTERVALS

3.1 Approach

This chapter presents a 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 extended kinematic equation-based formulae to calculate yellow change and red clearance intervals depending on movement type and approach speed. Not all potential aspects, practices, or elements of the process discussed in Chapter 2 were brought forward as recommendations; on those issues this chapter is silent. ITE recommended areas for further research in Section 2.20 as a result. Individual agencies may choose to extend their policies beyond the provisions in this recommended practice with appropriate engineering methods, procedures, documentation, and application of engineering judgment.

In addition to the MUTCD requiring the use of engineering methods,[163] agencies are encouraged to adopt a policies and procedures for establishing the method to calculate yellow change and red clearance intervals and to apply it consistently throughout their jurisdiction. The significant road-user benefit is derived by design consistency. The recommended application approach is to use primary data as a preferred choice; however, alternative approaches are noted for individual equations parameters based on research, engineering study or engineering judgment as application with a jurisdiction's policies and procedures.

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 from the start of the display of the steady red signal indication following the steady yellow signal indication until the display of a green signal indication to a conflicting traffic movement at a traffic signal. The purpose of the red clearance interval is to provide additional time for a vehicle legally in the intersection to leave the intersection before conflicting traffic movements begin.

3.3 General Requirements and Considerations

The *MUTCD*[164] states in Section 1A.02 that, "To be effective, a traffic control devices should meet five basic requirements: a) fulfill a need, b) command attention, c) convey a clear, simple meaning, d) command respect from road users, and e) give adequate time for a proper response." The following general requirements apply to the determination of yellow change and red clearance intervals based on Section 4D.26 of the 2009 *MUTCD*[165] 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 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*[166] 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 interval 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.

Rounding

Input parameters should be provided to level of accuracy of the data source and consistent with agency practices, preferably to the nearest 0.1 sec. of the value of parameter unless otherwise defined. Calculated interval values ending in 0.01 sec. to 0.09 sec. are recommended be rounded up to nearest 0.1 sec.

Where agencies use a look-up table of minimum yellow interval values associated with designated approach speeds (V_{85}) in 5 mph increments, measured 85th percentile speeds which fall between 5 mph increments should be rounded up to the next 5 mph increment. Agencies that use this approach should do so consistently throughout their jurisdiction.

3.4 Formula for Calculating Change and Clearance Intervals

The extended kinematic equation-based formulae should be used for calculating the minimum yellow change and the red clearance intervals with the approach speed input in mph and a unit conversion factor applied are as follows:

$$Y \geq t + \frac{1.47(V_{85} - V_E)}{a + 32.2g} + \frac{1.47V_E}{2a + 64.4g} \qquad \text{(U.S. units)} \quad \text{(A)}$$

$$R = \left[\frac{W + L}{1.47V_E}\right] - t_s \qquad \text{(U.S. units)} \quad \text{(B)}$$

Where:

Y = minimum yellow change interval (sec.);
t = perception-reaction time (sec.);
V_{85} = 85th percentile approach speed (mph);
V_E = intersection entry speed (mph);
a = deceleration (ft./sec./sec.);
g = grade of approach (percent/100, downhill is negative grade);
R = red clearance interval (sec.);
W = distance to traverse the intersection (width), stop line to far side no-conflict point along the vehicle path (ft.);
L = length of vehicle (ft.);
t_s = conflicting vehicular movement start up delay (sec.).

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

$$Y \geq t + \frac{0.28(V_{85} - V_E)}{a + 9.8g} + \frac{0.28V_E}{2a + 19.6g} \quad \text{(Metric units) (C)}$$

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

Where:

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

The extended kinematic equations reduces to the common form of the kinematic equation for through movements when V_{85} and V_E are assumed equal. The equations for the yellow change interval, Equations A and C, provide the minimum yellow change interval required to allow time for reasonable driver to see the yellow signal indication and decide whether to stop or to enter the intersection. If there is a grade on the approach to the intersection, these equations adjust the time to account for the gravitational acceleration caused by the slope of the road and its impact on the braking distance to be traversed.

This time includes the reasonable driver's perception-reaction time, generally 1.0 sec. Equations A and C provide reasonable drivers 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 yellow interval terminates. Yellow change intervals established based on the procedures of the Recommended Practice eliminate the dilemma zone, but there will always be an indecision zone because different drivers respond differently to the same set of circumstances. If the traffic signal is vehicular actuated, proper placement of the detection and controller settings can minimize the chances of a vehicle being in the indecision zone at the onset of the yellow change interval.

The equations for the red clearance interval, Equations B and D, provide a reasonable driver 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.

3.5 Application

Information on values for the inputs for calculating the change and clearance intervals for through movements at a signalized intersection 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.

Perception-Reaction Time, t

The minimum perception-reaction time is 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 increase this value. Note if the decision is made to modify the PRT, then the potential effect on deceleration should be reviewed.

85th Percentile Approach Speed, V_{85}

Through Movements

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 85th percentile approach speed should be measured on the intersection approach, upstream of the area of influence of the intersection operations. 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. Choice of data collection location and methodology should be determined by an agency's adopted practices and engineering judgment. The value of approach speed should not be less than the speed limit.

Further, please note that while the 2009 *MUTCD*[167]5 does not allow cycle-by-cycle changes in yellow change interval time (paragraph 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." (paragraph 13 Section 4D.26). This option is in the *MUTCD* because some old type controllers may not be able to be set for the same durations under different cycle lengths. 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 and substituted for V_{85} by the following equations for calculating the yellow change interval in U.S. units:

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

Where:

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

Or in metric units:

$$V_{85E} \text{ (through)} = SL + 11 \qquad \text{(Metric units)} \quad \text{(F)}$$

Where:

V_{85E} = estimated 85th percentile speed, in 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 and may adjust this estimation based on local conditions. In addition, where a signalized intersection has documented issues with red light running and enforcement methods are under consideration as potential countermeasures, agencies should collect primary data for through movement approach speed rather than rely on estimation. 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 and should not be less than the speed limit. 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.

Turning Movements

The yellow clearance interval values calculated by the extended kinematic equation model are significantly influenced by the values used for speeds in the equation. The approach speed for turning vehicles are the 85th percentile values under free flow conditions as measured through a speed study, or if known. Approach speed should be measured upstream of the intersection at the critical distance calculated for through movement vehicles or immediately upstream of the opening of the turn lane, whichever is closer. Accurate speed data are particularly important where approach speeds are greater than 40 mph, where turning vehicles are not required to enter a separate turn lane, or the separate turn lane is very long. Where actual speed data are not available, the speed limit should be used for approach speed.

Intersection Entry Speed, V_E

The actual 85th percentile intersection entry speeds measured through a speed study are recommended to be used. Intersection entry speed should be measured at the stop line of the intersection. Where the configuration of the intersection allows for higher intersection entry speeds, accurate speed data are especially important. Where actual speed data are not available, general values of 20 mph for intersection entry speed for left turns should be used.

Deceleration, a

A deceleration 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. Note if the decision is made to modify the deceleration rate, then the potential effect on PRT should be reviewed.

Approach Grade, g

Approach grades are measured on the intersection approach, from the location upstream of the area of influence of intersection operations used to define the 85th percentile approach speed through the far side intersection width clearance point and applied to all movements on that approach. The approach grade is measured to one decimal. 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. Alternative approaches for collecting data from electronic and on-line sources such as Google Earth™ may be used with appropriate citation. For new intersections, approach grade can be obtained from design plans. Where the grade changes over this distance or where grade information is not immediately available, engineering judgment should be used to estimate an appropriate value.

Minimums and Maximums

Through Movements

The recommended minimum value for the yellow change interval for through movements is 3.0 sec. and the recommended 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. There is no specified minimum or maximum value of the red clearance interval, if used.

Turning Movements

With the use of the extended kinematic equation model for left-turn movements, the recommendation is to use a minimum value of 3.0 sec. for the yellow change interval for left-turn movements with the allowance for the use of a maximum up to 7.0 sec. Care must be taken when using yellow change intervals greater than 6.0 sec. and the application of engineering judgment and/or study is necessary to address special road conditions. A specific minimum or maximum value of the red clearance interval for left turns is not suggested, if used.

Width of Intersection, W

Intersection width is the total distance to traverse the distance from the stop line to the curb-line extension, or outside edge of the travel lane, of farthest conflicting vehicular movement along the vehicle's travel path. Figure 3.1 illustrates intersection width for through movements and Figure 3.2 illustrates intersection width for left-turn movements. Where there are multiple lanes present for turning movements, 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 vehicular movement. This reference point is used, since the average pedestrian entry design time is 3.0 sec., during which a clearing vehicle will have traveled far beyond a crosswalk adjacent to the far-side curb extension. However, when there is frequent pedestrian traffic or the crosswalk(s) are protected by pedestrian signals, a longer distance for intersection width from the near-side stop line to include the far-side of departure crosswalk should be fully considered to improve safety of all users. However, intersection width is a

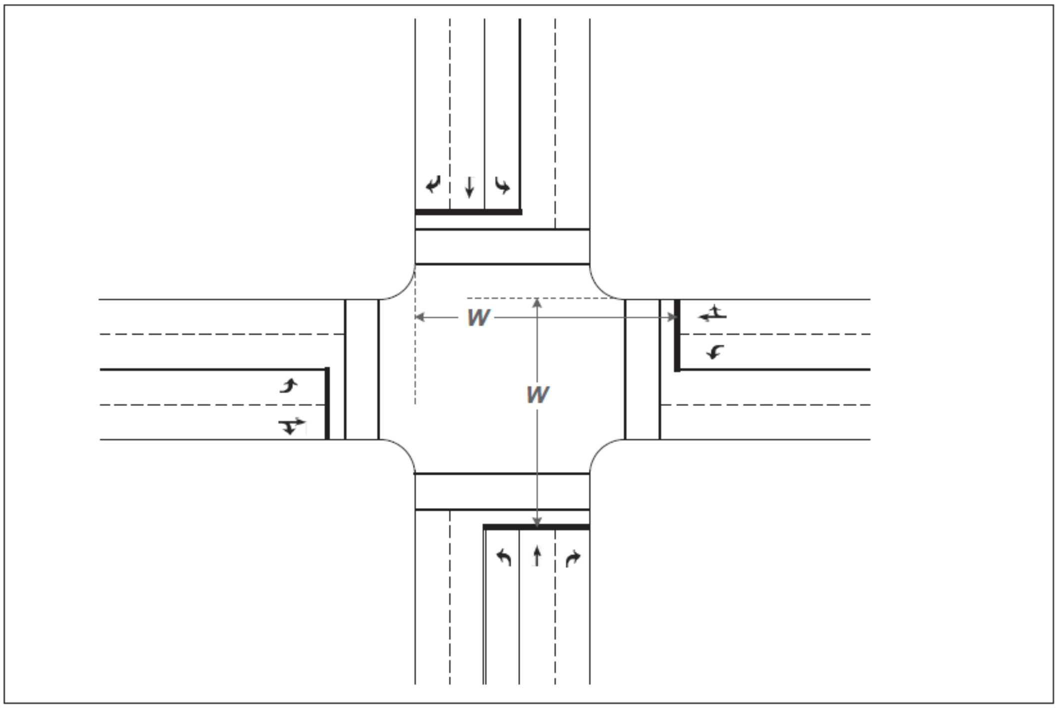

Figure 3.1: Diagram of Intersection Width Measurement for Through Movements

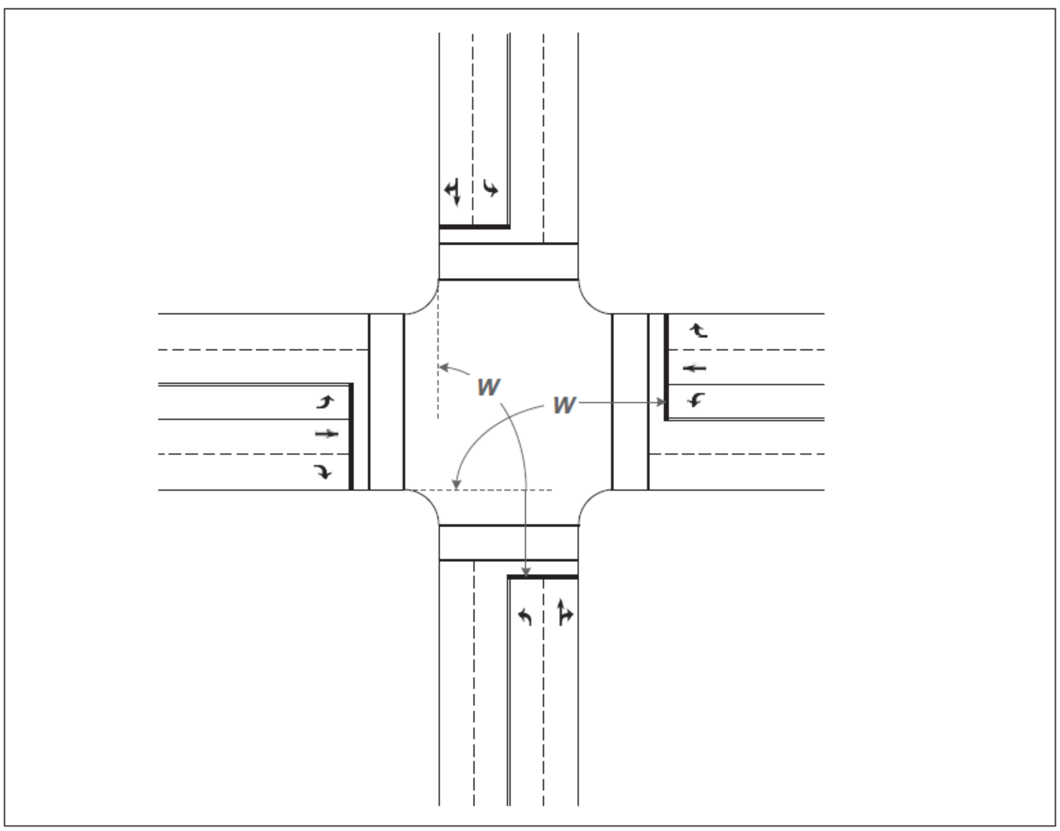

Figure 3.2: Diagram of Intersection Width Measurement for Left-Turn Movements

significant contributing variable to the duration of red clearance intervals and engineering judgment is necessary when considering longer intersection width definitions. Field measurements with an apparatus of choice provide the most accurate road width measure distance. 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 different vehicle length if 20 ft. is not representative of vehicles using the intersection. Different vehicle lengths may be considered based on a supporting vehicle classification study and application of engineering judgment.

Conflicting Vehicular Movement Start-Up Delay, t_s

Conflicting vehicular movement start-up delay is an optional parameter with an initial value set at 0.0 sec. Intersection entry delay values may be used based on engineering judgment or as supported by an engineering study. When applied an intersection entry delay factor is subtracted from the calculated red clearance interval, if used.

The engineer should use judgment when applying primary data to the calculation which may be necessary if the intersection is used regularly by bicyclists, has sharp turning radii, or the engineer determines a red clearance interval based on prevailing speed is not sufficient for the intersection.

Signal Phasing

The following notes the recommended approach to calculating the yellow change and red clearance intervals for different types of signal phasing for turning movements. While this approach may not take into account all possible turning 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.

Protected-Only Left-Turn Applications

Calculate the yellow change and red clearance intervals and adjust for grade if necessary for each approach and implement as calculated. The intervals can be of different durations for opposing approaches or adjacent through-movement phase. As noted above, in locations where approach speeds are higher (greater than 40 mph), where turning vehicles are not required to enter a separate turn lane or the separate turn lane is very long, and where the configuration of the intersection allows for higher intersection entry speeds, particular attention should be given to selecting appropriate approach and intersection entry speeds. The maximum yellow change interval should be limited to 7.0 seconds, regardless of the value calculated.

Permissive-Only Left-Turn Applications

Calculate the yellow change and red clearance intervals for the turning movements, adjust for grade if necessary, for opposing approaches, and for through movements, and select a yellow change interval value within the range calculated for both movements using engineering judgement based on volume of left turning traffic and overall intersection design (width, sight distance, safety performance, etc.). As noted above, in locations where approach speeds are greater than 40 mph, where turning vehicles are not required to enter a separate turn lane or the separate turn lane is very long, and where the configuration of the intersection allows for higher intersection entry speeds, particular attention should be given to selecting appropriate approach and intersection entry speeds. The maximum yellow change interval should be limited to 7.0 seconds regardless of the value calculated. 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 Applications

Calculate the yellow change and red clearance intervals and implement as described above for the respective protected and permissive portions of the phase.

Right- and Left-Turn Overlap Applications

A number of traffic signal phasing concepts use the concurrent phases, or overlaps, between a street's protected left-turn movement and the cross street's protected right-turn movement. Where a right-turn signal terminates concurrently with an overlapping left-turn signal, the longer of the change and clearance times would control. This method may be limited by the technical capabilities of the traffic signal controller.

Right-Turn Applications

There is limited research on the complex nature of driver behavior, interactions, and theoretical formulation for right-turn maneuvers and consequently, there are elements of these factors that are not fully understood. More information is necessary before making a definitive, separate recommendation for change and clearance intervals for right-turning vehicles. Thus, the recommended approach is to calculate the through movement and left-turn movement yellow change and red clearance interval times using the approaches defined in the individual cases described above.

3.6 Special Considerations

Across the breadth of signalized intersections there are designs such as alternative intersections and interchanges (SPUIs, LDIs, etc.) that present different geometric and traffic control scenarios than traditional intersections. Additionally, as bicycle traffic and dedicated bicycle lanes become more prevalent in the transportation system, this should be taken as part of the overall signal timing context. Lastly, there are overarching jurisdictional matters related to the underlying law for enforcement and restrictive vehicles codes.

Wide Intersections

What constitutes a wide intersection will vary regionally and individual agencies should define in the engineering practices the criteria for special consideration. For example, intersection widths of greater than 120 feet, signalized intersections of two multilane median divided highways, intersection width defined to the far-side of the departure crosswalk, long turning paths (typically for left turns) and single point diamond interchanges are potential examples of a wide intersections and may merit additional consideration. Specific methods adopted for wide intersection should be consistently applied based on supporting agency policy. Using the formulas, the engineer can calculate values for wider intersections. For wide intersections, this may result in long clearance intervals. The length of vehicle may be omitted from the calculation for the clearance interval in these cases. However, engineers should use their engineering judgment in the application of the formula for 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 line along the vehicle's path to the farthest point of conflicting traffic. The engineer may use 20 mph (32.2 km/h) as the 85th percentile speed of the turning vehicle or can collect the 85th percentile 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 may 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 is released if an engineering evaluation indicates a need for the increase. 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 for red clearance of 6.0 sec. Where there is an exclusive bicycle facility with separate bicycle signals, the engineer should refer to the most recent edition of the AASHTO *Guide for the Development of Bicycle Facilities*.

Restrictive Vehicle Codes

Variation in state 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. 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. In jurisdictions with restrictive laws that prohibit vehicles from being in the intersection when the light is red, the minimum yellow change interval should be longer than in jurisdictions with permissive laws.

Enforcement

This Recommended Practice is intended to support the development of safe and appropriate change and clearance intervals and does not cover enforcement actions. However, since several of the variables used in the recommended procedures for calculating yellow change intervals actually represent a range of driver behaviors and other factors, it is important to note that actions to enforce red light violations, either through traditional or automated means, with zero tolerance are not appropriate.

3.7 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.

3.8 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 the following:

- 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 *MUTCD*.
- 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 *NCHRP Report 812: Signal Timing Manual, Second Edition*[168] 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.

APPENDIX

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

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) =

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

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

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

END NOTES

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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-based formulae 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 and valuable 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.

Interval Sequence—the order of appearance of signal indications during successive intervals of a signal cycle.

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—A timing unit associated with the control of one or more movements. Phases are often assigned to vehicular and pedestrian 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.

Phase Sequence—The order of phases in a sequence structure (e.g., ring) consisting of two or more sequentially timed and individually selected conflicting movements, arranged to allow flexibility between compatible movements in different sequence structures (e.g., rings).

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 line 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 line will decide to stop if she or he believes there is sufficient distance within which to do so comfortably.

Red Clearance Interval—An interval following the yellow change interval and preceding the next conflicting green interval during which conflicting traffic movements at an intersection view a red signal indication and are not permitted to enter the intersection. It allows time for vehicles that 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—A pavement marking that denotes where traffic should stop in advance of an intersection (sometimes referred to as a Stop Bar).

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 from which they are approaching and departing an 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 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.

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