TRAFFIC ENGINEERING HANDBOOK

SEVENTH EDITION

Institute of Transportation Engineers

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Table 10-4. Minimum Green Requirements to Satisfy Queue Clearance

Distance Between Stop Line and Nearest Upstream Detector, ft	Minimum Green Needed to Satisfy Queue Clearance[1, 2] (G_q), s
0 to 125	5
26 to 50	7
51 to 75	9
76 to 100	11
101 to 125	13
126 to 150	15

Notes:

[1] Minimum green values listed apply only to phases that have one or more advance detectors, no stop line detection, and the added initial parameter is not used.

[2] Minimum green needed to satisfy queue clearance, $G_q = 3 + 2n$ (in seconds), where n = number of vehicles between stop line and nearest upstream detector in one lane. And, $n = Dd/25$, where Dd = distance between the stop line and the downstream edge of the nearest upstream detector (in feet) and 25 is the average vehicle length (in feet), which could vary by area.

Source: Koonce et al. (2008), Table 5-4, pp. 5-12.

time would be the pedestrian walk interval plus pedestrian crossing clearance interval (estimation described later in this section).

2. *Maximum Green*

The maximum green parameter is used to terminate a phase based on a set maximum amount of time that a green signal indication can be displayed if conflicting demand has been detected. Maximum green is used to limit the delay to other movements at the intersection and to keep the cycle length from exceeding a maximum amount. It also protects the signal from failing to serve the demand from different phases in the case of detector failures (Koonce et al., 2008).

The *Traffic Signal Timing Manual* (Koonce et al., 2008) recommends two methods for determining the maximum green time. Both methods estimate the green duration needed for average volume conditions and inflate this value to accommodate cycle-to-cycle peaks. Details of the methods may be found in the *Traffic Control Devices Handbook* (Seyfried, 2013). One of the methods for estimation of maximum green time requires establishing an equivalent optimal pre-timed signal timing plan using critical movement analysis. The details of this analysis may be found in Chapter 3 of the *Traffic Signal Timing Manual* (Koonce et al., 2008). The essential idea is to set the green interval for each phase in proportion to the critical lane group volume for each phase. *Critical lane group* is the lane group with most intense demand (and not necessarily with the highest volume). For example, a lane group with many left-turning vehicles may have more intense demand than a higher-volume lane group that only serves through vehicles (Roess, Prassas, & McShane, 2004). The pre-timed plan can be optimized using traffic simulation software packages.

3. *Vehicle Extension*

Vehicle extension (also referred to as *passage time, passage gap*, or *unit extension*) extends the green interval based on the detected vehicles once the phase is green. This parameter extends the green interval for each vehicle actuation up to the maximum green. In a coordinated signal system (discussed later in this chapter), the vehicle extension period is also subject to termination by Force Off.

4. *Yellow Change Interval*

The purpose of a yellow signal indication is to warn approaching traffic of an imminent change in right-of-way assignment. Therefore, it warns that the related green movement is ending and/or a red indication will be displayed immediately thereafter. The yellow change interval has a predetermined duration calculated through engineering practices.

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, so long as the driver entered before or during the yellow signal indication. Jurisdictions with permissive laws may use a red clearance interval to ensure that 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 (46 U.S. states and 12 Canadian provinces). Under *restrictive laws*, drivers may not enter the intersection during the yellow signal indication unless the intersection can be cleared prior to the onset of the red indication or unless it is impossible or unsafe to stop (4 U.S. states).

The motorist's decision to decelerate to a stop is based on the perceived distance from the intersection for the speed traveled, and on his/her experience with braking. At a theoretical critical point, a motorist may decide to either brake to a stop or proceed. The duration of a yellow change interval provides enough yellow time for a vehicle to travel, starting with an initial approach speed, over the distance it would take to stop at a comfortable average deceleration before entering the intersection (Eccles & McGee, 2001). Based on this, the yellow change interval for a given speed is determined by driver perception–reaction time (PRT), approach speed, and vehicle deceleration rates. A PRT of one second is considered an adequate value for most drivers. A braking deceleration rate of 10 ft/sec/sec (3.0 m/sec/sec) is considered comfortable by the greater majority of motorists. Many motorists may be willing to brake at a slightly less comfortable rate, corresponding to a value greater than 10 ft/sec/sec (3.0 m/sec/sec), while a few prefer a lower rate. The selection of these discrete values will tend to accommodate the needs of most motorists and results in a conservative design.

The following equation in U.S. units provides a theoretical basis for the calculation of yellow change interval.

$$Y = t + \frac{1.47V}{2a + 2Gg} \quad \text{(U.S. units)} \tag{10-1}$$

where,

Y = length of the yellow change interval (sec)
V = 85th percentile approach speed (mph)
t = perception–reaction time, generally assumed as 1.0 sec
a = average deceleration rate, generally assumed as 10 ft/sec/sec
g = approach grade (percent divided by 100, negative for downgrade)

The model was initially proposed in an ITE report, *Determining Vehicle Signal Change and Clearance Intervals* (Thompson, 1994), and is widely known as the *ITE formula* and a guideline for yellow interval determination. Engineering practices for determining the duration of the yellow change interval were published in ITE's *Manual of Traffic Signal Design* (Kell & Fullerton, 1991). The history of the yellow change interval computation was further explored in the 2001 ITE publication, *A History of the Yellow and All-Red Intervals for Traffic Signals* (Eccles & McGee, 2001).

The *MUTCD* provides guidance that the yellow change interval should range between 3.0 and 6.0 sec. At intersections with downhill approaches, the related gravitational forces require greater braking distances and longer yellow change intervals. In contrast, uphill approaches require lesser braking distances and shorter yellow change intervals.

A recent study by the National Cooperative Highway Research Program (NCHRP; McGee et al., 2012) comprehensively reviewed the current practice on timing of yellow change and red clearance intervals, and conducted various field studies at a number of signalized intersections nationwide (McGee et al., 2012).The methods used for timing the yellow interval, which were reviewed in that study, include the kinematic equation, rule of thumb, uniform value, stopping probability, combined kinematic and stopping probability, and modified kinematic equation for left-turn movements. Study results have shown that modifying yellow change intervals to the duration calculated by the ITE formula can reduce red-light running between 36 and 50%.

The NCHRP study also gives recommendations for the parameter values. Based on field observations, the mean perception–reaction time was found to be 1.0 sec, and the mean deceleration rate was found to be 10 ft/sec/sec (3 m/sec/sec). Both are the generally accepted values used in the ITE formula. For the 85th percentile approach speed, the study found that speed limit can be an inaccurate estimate. The study suggested that the 85th percentile approach speed for through movements can be estimated by adding 7 mph (11 km/h) to the approach speed limit. The 85th percentile approach speed for left-turn movements can be estimated by subtracting 5 mph (8 km/h) from the approach speed limit.

Practicing traffic engineers may encounter unique situations that warrant modifying the parameters discussed herein. Engineering judgment may be applied in those circumstances and the modifications should be documented along with supporting information justifying them.

5. *Red Clearance Interval*

As previously discussed, the duration of the yellow change interval is set to ensure that motorists are able to enter the intersection prior to the termination of the yellow change interval. Motorists far downstream of the critical point at the onset of the yellow change interval will either be well within or totally clear of the intersection when the yellow change interval ends. However, some motorists who are just past the critical point on the approach to the intersection when the yellow change interval begins might just barely cross the stop line when the yellow change interval ends. Thus, traffic on the cross-street needs to be released only after these motorists clear any possible conflicts. To do this, the red clearance interval is introduced following the end of the yellow change interval, during which the phase of the cross-street has a red signal display before the display of a green signal. The red clearance interval is also known as the *all-red interval.* It can partially or fully clear motorists who are proceeding through the intersection at the end of the yellow change interval. It may also be used to help clear vehicles that are queued within the intersection because of the lack of gaps for permissive left turns or other reasons.

The duration of the red clearance interval can be set to provide full or partial clearance. Full clearance comprises the width of the intersection, possibly including near-side and far-side crosswalks, plus the length of the vehicle. The ITE publication *A History of Yellow and All-Red Intervals for Traffic Signals* (Eccles & McGee, 2001) provides the evolution of the equation for calculating the red clearance interval for full clearance.

The following equation adapts the formula to U.S. units allowing the use of velocity (V) in mph:

$$R = \left[\frac{W + L}{1.47V}\right] \quad \text{(U.S. units)} \tag{10-2}$$

where,

R = Red clearance interval (sec)
V = 85th percentile approach speed (mph)
L = Vehicle length, generally assumed to be 20 ft
W = Intersection width (ft)

The NCHRP study (McGee et al., 2012) reviewed methods for red clearance interval timing, including the kinematic equation, uniform value, conflict zone, and modified kinematic equation for left-turn movements. The study concluded that calculating the durations of red clearance intervals using the ITE equation has been shown to reduce total crashes between 8 and 14%, while reducing injury crashes by approximately 12% and increasing the red clearance interval to the duration calculated by the ITE equation has not shown to increase red-light running events.

The NCHRP study evaluated the start-up delay of conflicting traffic through field observations. An average value of 1.1 sec was found in the study and thus it recommended that 1.0 sec be subtracted from the ITE red clearance interval equation to account for this factor. The NCHRP study also gives recommendations for the parameter values in the red clearance interval equation. For the 85th percentile approach speed, it was found that speed limit is an inaccurate estimate. It is suggested that the 85th percentile approach speed for through movements can be estimated by adding 7 mph (11 km/h) to the approach speed limit. Furthermore, the study recommends that the minimum red clearance interval be 1.0 sec.

When calculating the red clearance interval for left-turn movements, the study suggests that 20 mph (32 km/h) can be used regardless of the posted speed limit. The vehicle length is suggested to be 20 ft (6 m), which is the generally accepted value for passenger cars. Increasing the length to accommodate larger vehicles was not considered necessary. The intersection width is recommended to be the distance from the upstream edge of the nearside stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel lane. For left-turning vehicles, the measurement would be along the turning path.

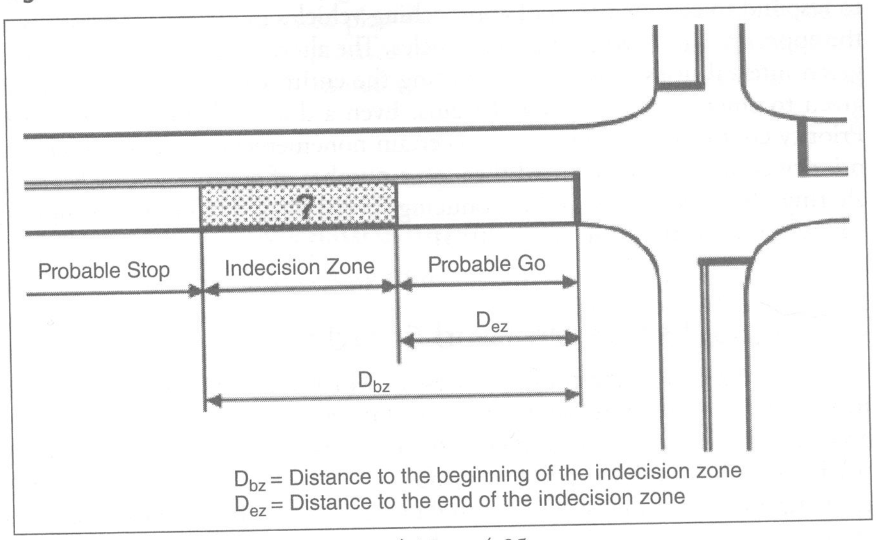
When there are unique conditions that may warrant modifying the parameters, engineering judgment may be applied and documented with supporting information justifying the modifications.

6. *Dilemma Zone*

On high-speed approaches, a road user's indecisive behavior gives rise to an indecision zone (also referred to as Type II dilemma zone). Drivers within a few seconds' travel time from the intersection STOP bar at the onset of the yellow indication tend to be indecisive about their ability to stop. Figure 10-14 provides a depiction of this zone.

If the driver plans to proceed through the intersection but the conflicting approach becomes green, a severe right-angle crash may result. The other option, an abrupt stop, involves the risk of a rear-end collision. Chapter 10 of the *Traffic Control Devices Handbook* (Seyfried, 2013, pp. 363–365) provides details of the methods for dilemma zone protection. These methods involve fully actuated control with extra detection capabilities in advance of the STOP bar at the high-speed intersection approach. In a semi-actuated signal (which does not have detectors on the major street), dilemma zone protection cannot be implemented. Hence, semi-actuated signal control should not be implemented when the high-speed approaches are the non-actuated phase.

Figure 10-14. Dilemma or Indecision Zone



Source: Koonce et al. (2008), Figure 4-18, p. 4-25.

7. *Pedestrian Timing Intervals*

The pedestrian phase consists of three intervals: WALK; pedestrian change, commonly referred to as flashing don't walk (FDW); and steady DON'T WALK. The first of the three intervals is indicated by the white WALKING PERSON (symbolizing WALK) on the pedestrian signal. The pedestrian change interval follows the walk interval and the flashing orange UPRAISED HAND (symbolizing flashing DONT WALK) is displayed. The steady DON'T WALK interval follows the pedestrian change interval and is indicated by a steady orange UPRAISED HAND indication. When the steady UPRAISED HAND is displayed, conflicting vehicular phases are initiated and served (FHWA, 2009).

8. *Walk Interval*

The walk interval should provide pedestrians adequate time to perceive the WALK indication and depart the curb before the pedestrian clearance interval begins. The *MUTCD* (FHWA, 2009) indicates that the minimum walk duration should be at least 7 seconds, but indicates that duration as low as 4 seconds may be used if pedestrian volumes are low or pedestrian behavior does not justify the need for 7 seconds. In urban situations with significant levels of pedestrian traffic, it may be necessary to use a longer walk interval in order to provide the time for the queue of pedestrians to begin their crossing by leaving the curb before the end of the walk interval. Consideration should be given to walk durations longer than 7 seconds in school zones and areas with large numbers of elderly pedestrians. In cases where the pedestrian push button is a considerable distance from the curb, additional WALK time may be desirable. Ultimately, the duration of the walk interval is established by local agency policy within the range of guidance provided in the *MUTCD* and the Americans with Disabilities Act. The length of the walk interval must be sensitive to specific conditions at each location rather than "one size fits all."

9. *Pedestrian Clearance Time*

Pedestrian clearance time is computed as the crossing distance divided by the walking speed. The *MUTCD* (FHWA, 2009, Section 4E.06) recommends walking speed for calculating the pedestrian clearance time to be 3.5 ft/sec. A slower 3.0 ft/sec walking speed is also indicated for use as a "cross-check" calculation (Paragraph 14 of Section 4E.06 of the *MUTCD*) to determine if there is sufficient crossing time for slower pedestrians, such as those in wheelchairs or who are visually disabled, to cross wide streets. The evolution of the guidelines on walking speed was discussed in Chapter 3 of this resource. A walking speed of up to 4 ft/sec may be used to evaluate the sufficiency of the pedestrian clearance time at locations where an extended pushbutton press function has been installed to provide slower pedestrians an opportunity to request and receive a longer pedestrian clearance time through accessible pedestrian detectors. All pedestrian signal heads used at crosswalks where the pedestrian change interval is more than 7 seconds shall include a pedestrian change interval countdown display in order to inform pedestrians of the number of seconds remaining in the pedestrian change interval (FHWA, 2009).