However, if the spacing between signals is not consistent, the spacing is relatively short, or the cycle length is relatively long, perfect progression in both directions is not possible. In this situation, progression in one direction is usually possible. It may then be desirable to provide perfect progression in one direction and non-progressive flow in the other, if the traffic is highly directional. However, if not, the offsets can be set for semi-progressive flow in both directions. In less than ideal situations, thoughtful compromises must be made. In network systems, progressive traffic flow on multiple streets that cross one another is a complex challenge, unless the signal spacing is ideal in all directions. The operation of semi-actuated signals at poor timing points, if feasible, can help to minimize disruptions to progressive traffic flow.

Along short reaches of arterial streets, the offsets can be evaluated and determined by drawing a time-space diagram. In such a diagram, the sequential green and red intervals are plotted on one axis (usually vertical) at each signalized location and the signal spacing on the other axis (usually horizontal). The slope between the two axes represents the progressive design speed. However, for longer arterial systems or network systems, computer programs can be used to optimize a selected objective function. These programs examine vehicle arrivals, queues, and discharges and can calculate total stops, delays, and other factors.

Where signals are widely spaced or where volume is low, traffic tends to arrive randomly rather than in distinct platoons. Thus, there would be no need for offsets under these conditions. Generally, signals are considered to be widely spaced when they are 1 mi. (1.6 km) or more apart. Volume may be considered to be too low for coordinated signal timing when the two-way hourly volume divided by the distance in feet is less than 0.5 (less than 1.6 when divided by the distance in meters). Accordingly, in some systems the signals operate "free" until a specific volume threshold is detected, thus activating coordinated operation using offsets.

Timing Plans

A signal timing plan consists of a unique combination of a cycle length, offset, and splits. Modern controllers offer multiple combinations of these parameters.

The number of timing plans required for efficient operation is governed by the diverse traffic patterns. Patterns can be identified and categorized by examining and comparing the volumes on the various approaches. Typical patterns include the following periods:

- A.M. peak;
- mid-day;
- P.M. peak;
- late evening;
- weekend; and
- special events.

In the more sophisticated systems in large urban areas, there can be refinements to the above categories. Currently, in most systems plans are implemented at a predetermined time—for example, 3:30 p.m. for the P.M. peak period. "Traffic responsive" systems automatically implement plans based on volume and/or occupancy thresholds being reached rather than by time of day. Further, "traffic adaptive" systems automatically implement plans on a more dynamic basis, as frequently as every cycle. These systems use optimization algorithms to determine cycle lengths, splits, and offsets.

Green Intervals and Splits

The duration of each green interval generally should be set in proportion to the critical lane volume for each phase, after consideration of pedestrian accommodation. For example, consider a simple two-phase intersection where the arterial street accommodates 2,000 veh./hr. with four lanes and the minor street accommodates 500 veh./hr. with two lanes, both in their peak directions. The critical lane volume for the arterial street is 500 veh./hr./lane while the critical lane volume for the side street is 250 veh./hr./lane. Thus, the green interval for the arterial street should be twice that of the minor street or two-thirds of the available green time within the cycle length during that period. If the signal is operated as pre-timed, the split, or proportionate green time, can be readily identified. If the signal is operated as semi-actuated, the average green time used for the minor street should be approximately one-third of the cycle length. If the signal is operated as full-traffic actuated, the maximum green settings for both streets generally should reflect the desired ratio.

The volume-per-lane ratio method discussed above is adequate for most intersections. However, at multi-phase locations or at locations operating near capacity, a more precise method can be used, which examines volume-to-capacity ratios. The most recent version of the *Highway Capacity Manual* can be used to obtain volume-to-capacity ratios. Using programs like Highway Capacity Software (HCS), various green interval lengths can be simulated to determine which optimize the volume-to-capacity ratio for an isolated intersection. Ideal optimized timing would result in the volume-to-capacity ratios being approximately equal for each of the critical lane groups.

Yellow Change Interval

The purpose of a yellow signal indication is to warn approaching traffic of an imminent change in right-of-way assignment. As such, it warns 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.

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 approach initial speed, over the distance it would take to stop at a comfortable average deceleration before entering the intersection.[52] 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 equations in U.S. and metric units provide theoretical basis for the calculation of yellow change interval.

$$Y = t + \frac{1.47\,V}{2a+64.4g} \ (\textit{U.S. units}) \qquad Y = t + \frac{0.28\,V}{2a+19.6g} \ (\textit{Metric units}) \quad (2)$$

Where,

Y = length of the yellow change interval (sec);

V = 85th percentile approach speed (mph or km/h);

t = perception-reaction time, generally assumed as 1.0 sec;

a = average deceleration rate, generally assumed as 10 ft./sec./sec. (3.0 m/sec./sec.); and,

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*,[53] 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*.[54] 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*.[55]

Table 10-10 provides the yellow change intervals for different approach speeds based on the calculation from Equation 2. All the yellow change intervals in Table 10-10 are for level terrain (0 percent grade). 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. On the other hand, uphill approaches require lesser braking distances and shorter yellow change intervals.

A recent study by the National Cooperative Highway Research Program (NCHRP) 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.[56] The methods used for timing the yellow interval, which have been reviewed in that study, include the kinematic equation, "rule-of-thumb," uniform value, stopping probability, combined

TABLE 10-10. Yellow Change Interval for Different Approach Speeds (0% grade)

Approach Speed (v) (mph)	Approach Speed (v) (ft./sec.)	Yellow Change Interval (sec.)
25	36.7	3.0[a]
30	44.0	3.2
35	51.3	3.6
40	58.7	3.9
45	66.0	4.3
50	73.3	4.7
55	80.7	5.0
60	88.0	5.4
Approach Speed (v) (km/h)	**Approach Speed (v) (m/sec.)**	**Yellow Change Interval (sec.)**
40	11.2	3.0[a]
50	14.0	3.3
60	16.8	3.8
70	19.6	4.3
80	22.4	4.7
90	25.2	5.2
100	28.0	5.7

a. The 2009 Edition of the MUTCD with Revision Numbers 1 and 2 incorporated, dated May 2012 recommends a minimum duration of 3 seconds for the yellow change interval.

kinematic and stopping probability, and modified kinematic equation for left-turn movements. Study results have shown modifying yellow change intervals to the duration calculated by the ITE formula, as indicated by Equation 2, can reduce red-light running between 36 and 50 percent.

The NCHRP study also gives recommendations for the parameter values in Equation 2. Based on field observations, the mean perception-reaction time was found to be 1.00 sec., and the mean deceleration rate was found to be 10.08 ft./sec./sec. (3.07 m/sec./sec.). Both are the generally accepted values used in the ITE formula. For 85th percentile approach speed, it was found that speed limit can be an inaccurate estimate. It is suggested 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.

Finally, the NCHRP study proposed the following guidelines for timing the yellow change interval:

- The yellow change interval (Y) is calculated using the ITE Equation (i.e., Equation 2);
- the perception-reaction time (t) is 1.0 sec.;
- the deceleration rate (a) is 10 ft./sec./sec. (3.0 m/sec./sec.);
- the approach speed (V) is the 85th percentile speed in mph or km/h determined under free-flow conditions. If the 85th percentile approach speed is unavailable, it can be estimated as the posted speed limit plus 7 mph (11 km/h), see Table 10-11; and
- for left-turn movements, the approach speed (V) should be set at the approach speed limit minus 5 mph (8 km/h).

Red Clearance Interval

As previously discussed, the duration of the yellow change interval is set to ensure 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 (using the values shown in Table 10-11). 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* provides the evolution of equation for calculating the red clearance interval for full clearance.

TABLE 10-11. Yellow Change Interval by Approach Speed Limit and Grade Provided by NCHRP Study

Posted Speed Limit (mph)	Grade (%) −4	−2	0	2	4
25	3.7	3.5	3.4	3.2	3.1
30	4.1	3.9	3.7	3.6	3.4
35	4.5	4.3	4.1	3.9	3.7
40	5.0	4.7	4.5	4.2	4.1
45	5.4	5.1	4.8	4.6	4.4
50	5.8	5.5	5.2	4.9	4.7
55	6.2	5.9	5.6	5.3	5.0
Posted Speed Limit (km/h)	**Grade (%) −4**	**−2**	**0**	**2**	**4**
50	3.7	3.5	3.3	3.2	3.1
60	4.2	4.0	3.8	3.6	3.5
70	4.8	4.5	4.3	4.1	3.9
80	5.3	5.0	4.7	4.5	4.3
90	5.8	5.5	5.2	4.9	4.7
100	6.0[a]	6.0	5.7	5.4	5.1

NOTE: Yellow change intervals calculated using 85th percentile approach speed estimation of posted speed limit +7 mph (+11.3 km/h).

a. The 2009 Edition of the MUTCD with Revision Numbers 1 and 2 incorporated, dated May 2012 recommends a maximum duration of 6 seconds for the yellow change interval.

SOURCE: Adapted from McGee, H. Sr., et.al. NCHRP Report 731 *Guidelines for Timing Yellow and Red Intervals at Signalized Intersections.* Washington, DC: National Cooperative Highway Research Program, 2012.

Equation 3 adapts the formula to U.S. and metric units allowing the use of velocity (V) in mph or km/h.

$$R = \frac{W+L}{1.47\,V} \;(\textit{U.S. units}) \qquad R = \frac{W+L}{0.28\,V} \;(\textit{Metric units}) \qquad (3)$$

Where,

R = Red clearance interval (sec.);

V = Approach speed (mph or km/h);

L = Vehicle length, generally assumed to be 20 ft. (6 m); and

W = Intersection width (ft. or m).

Table 10-12 shows full red clearance intervals for various intersection widths.

Some agencies prefer to use shorter red clearance intervals than those shown in Table 10-12, thus providing partial clearance before the start of the cross street green. The rationale for partial clearance is that cross street traffic is delayed due to the start-up delay at the start of their green. If a reaction time of 1.0 sec is used, the red clearance interval for partial clearance, due to delay, R_D, is calculated using Equation 4:

$$R_D = \frac{W+L}{1.47\,V} - 1 \;(\textit{U.S. units}) \qquad (4)$$

$$R_D = \frac{W+L}{0.28\,V} - 1 \;(\textit{Metric units})$$

TABLE 10-12. Red Clearance Interval for Various Approach Speeds and Intersection Widths

Approach Speed (v)		Width of Intersection (ft.) 30	50	70	90	110
(mph)	(ft./sec.)	Red Clearance Interval (sec.)				
25	36.7	1.4	1.9	2.5	3.0	3.5
30	44.0	1.1	1.6	2.0	2.5	3.0
35	51.3	1.0	1.4	1.8	2.1	2.5
40	58.7	0.9	1.2	1.5	1.9	2.2
45	66.0	0.8	1.1	1.4	1.7	2.0
50	73.3	0.7	1.0	1.2	1.5	1.8
55	80.7	0.6	0.9	1.1	1.4	1.6
60	88.0	0.6	0.8	1.0	1.2	1.5
Approach Speed (v)		**Width of Intersection (m) 9.1**	**15.2**	**21.3**	**27.4**	**33.5**
(km/h)	(m/sec.)	Red Clearance Interval (sec.)				
40	11.2	1.4	1.9	2.4	3.0	3.5
50	14.0	1.1	1.5	2.0	2.4	2.8
60	16.8	0.9	1.3	1.6	2.0	2.4
70	19.6	0.8	1.1	1.4	1.7	2.0
80	22.4	0.7	0.9	1.2	1.5	1.8
90	25.2	0.6	0.8	1.1	1.3	1.6
100	28.0	0.5	0.8	1.0	1.2	1.4

SOURCE: Adapted from Koonce, P., et al., *Traffic Signal Timing Manual,* Washington, DC: Federal Highway Administration Report No. FHWA-HOP-08-024, June 2008, http://www.signaltiming.com/.

In the recent NCHRP study[58] which comprehensively reviewed the current practice on timing of yellow change and red clearance intervals, methods for red clearance interval timing including the kinematic equation, uniform value, conflict zone, and modified kinematic equation for left- turn movements have been reviewed by various field studies. The study concluded calculating the durations of red clearance intervals using the current ITE guidelines (i.e., Equation 3) has been shown to reduce total crashes between 8 and 14 percent while reducing injury crashes by approximately 12 percent; and increasing the red clearance interval to the duration calculated by current ITE guidelines (i.e., Equation 3) has not shown to increase red-light running events.

When the start-up delay of the conflicting traffic is considered when timing the red clearance interval (i.e., using Equation 4), the NCHRP study also evaluated the start-up delay through field observations. An average value of 1.1 sec. was found in that study, which validates Equation 4 where 1 sec. is used.

The NCHRP study also gives recommendations for the parameter values in Equations 3 and 4. For 85th percentile approach speed, it was found that speed limit is an inaccurate estimate. It is suggested the 85th percentile approach speed for through movements can be estimated by adding 7 mph (11 km/h) to the approach speed limit (see Figure 10-13).

When calculating the red clearance interval for left-turn movements, it is suggested 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 is 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.

Finally, the NCHRP study proposed the following guidelines for timing the red clearance interval:

- the red clearance interval (R) is calculated using Equation 4;
- the intersection width (W) should 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 outside edge of the farthest travel lane;
- the length of vehicle (L) is set at 20 ft. (6 m); and,
- the approach speed (V) is the same approach speed used to calculate the yellow change interval for through movements.

For left-turn movements, the following modification of the parameter values should be made:

- The approach speed (V) should be set at 20 mph (32 km/h) regardless of the approach speed limit; and
- the width of the intersection (W) should be the length of the approaching vehicle's turning path measured from the upstream edge of the approaching movement stop line to the far side of the intersection cross street, as defined by

TABLE 10-13. Red Clearance Interval by Posted Approach Speed Limit and Intersection Width Provided by NCHRP Report 731

	Width of Intersection (ft.)				
Posted Speed Limit	30	50	70	90	110
(mph)	Red Clearance Interval (sec.)				
25	1.0	1.0	1.0	1.3	1.8
30	0.0	1.0	1.0	1.0	1.4
35	0.0	1.0	1.0	1.0	1.1
40	0.0	1.0	1.0	1.0	1.0
45	0.0	0.0	1.0	1.0	1.0
50	0.0	0.0	1.0	1.0	1.0
55	0.0	0.0	0.0	1.0	1.0
60	0.0	0.0	0.0	1.0	1.0
	Width of Intersection (m)				
Posted Speed Limit	9.1	15.2	21.3	27.4	33.5
(km/h)	Red Clearance Interval (sec.)				
40	1.0	1.0	1.0	1.3	1.8
50	0.0	1.0	1.0	1.0	1.3
60	0.0	1.0	1.0	1.0	1.0
70	0.0	0.0	1.0	1.0	1.0
80	0.0	0.0	1.0	1.0	1.0
90	0.0	0.0	0.0	1.0	1.0
100	0.0	0.0	0.0	1.0	1.0

NOTE: Yellow change intervals calculated using 85th percentile approach speed estimation of posted speed limit +7mph (+11.3 km/h).

SOURCE: McGee, H. Sr., et. al. NCHRP Report 731—*Guidelines for Timing Yellow and Red Intervals at Signalized Intersections*. Washington, DC: National Cooperative Highway Research Program, 2012.

the extension of the curb line or outside edge of the farthest travel lane.

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

Pedestrian Intervals

The design and operation of traffic signals must consider the needs of pedestrians. Where pedestrians are to be accommodated, signal indications visible to them are required. In cases where they may proceed with concurrent vehicular traffic, a visible vehicle signal face can meet minimum pedestrian needs. However, vehicular signal faces provide pedestrians no information as to when there is insufficient time left to complete a crossing. Accordingly, pedestrian signal heads, consisting of the WALKING PERSON symbol, and the UPRAISED HAND symbol, and pedestrian countdown signals, are preferable to vehicular signal indications when designs for new signals and signal upgrades are being considered.

The WALKING PERSON symbol identifies the time available to a pedestrian to initiate a crossing or to enter the crosswalk. This period is called the Walk interval. Generally, the Walk interval overlaps with part of the concurrent Green interval for vehicle traffic, and should be no less than 7 sec., although in rare cases it may be as brief as 4 sec. Where the cycle length permits, a Walk interval of 10 sec. generally should be used and should be adequate for most situations. Where the cycle length does not permit 10 sec., or where pedestrian crossings are infrequent, a lower duration may be used. In high-volume pedestrian areas, such as near schools, colleges, retail districts, sports venues, and entertainment areas, longer intervals may be required to allow all waiting pedestrians to enter the crosswalk before the Walk interval concludes. At these types of locations, site studies should be conducted to document pedestrian queuing and to determine the required duration of the Walk interval. Table 10-14 lists typical pedestrian Walk intervals recommended by the *Traffic Signal Timing Manual*.[59]

A flashing UPRAISED HAND signal indication means a pedestrian shall not start to cross the roadway in the direction of the signal indication, but any pedestrian who has already started to cross on a steady WALKING PERSON signal indication shall proceed out of the traveled way. The pedestrian clearance interval commences at the onset of the flashing UPRAISED HAND and concludes with a steady UPRAISED HAND, generally at the end of the green interval, or in some cases at the end of the yellow interval.

The MUTCD requires pedestrian countdown signals to be used except when the pedestrian change interval is 7 sec. or less. Additionally, the countdown is not to begin until the start of the flashing UPRAISED HAND indication. This is because in vehicle-actuated systems that use the "rest in walk" feature with a variable-duration vehicular green phase, it is not feasible to display a countdown during the Walk interval. While the vehicular phase is either "resting" (with no vehicles detected on conflicting phases) or being extended by approaching vehicles, the parallel concurrent pedestrian phase remains in WALKING PERSON. In the absence of a conflicting call, the WALKING PERSON indication remains on indefinitely. It is only after a conflicting phase call is detected the pedestrian change interval (flashing UPRAISED HAND) begins timing. With this "rest-in-walk" operation, it is not feasible to count down the Walk interval. Even though some jurisdictions do not use the "rest in walk" mode, and some may have virtually all pre-timed signals, it would be confusing to pedestrians if they encountered different countdown operations at different intersections, within the same jurisdiction or as they travel from one jurisdiction to another. The common denominator workable with all signals, regardless of actuated or pre-timed, is to count down only the pedestrian change interval.[60]

TABLE 10-14. Typical Pedestrian Walk Interval

Conditions	Walk Interval Duration (*PW*), s
High pedestrian volume areas (e.g., school, central business district, sports venues, etc.)	10 to 15
Typical pedestrian volume and longer cycle length	7 to 10
Typical pedestrian volume and shorter cycle length	7
Negligible pedestrian volume	4
Conditions where older pedestrians are present	Distance to center of road divided by 3.0 feet per second

SOURCE: Koonce, P., et al. *Traffic Signal Timing Manual*, Washington, DC: Federal Highway Administration Report No. FHWA-HOP-08-024, June 2008, http://www.signaltiming.com/.