TRAFFIC SIGNAL TIMING GUIDELINES & TRAINING MANUAL

Interim Edition
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**GLOSSARY**

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1. INTRODUCTION

Purpose

The following guidelines and recommendations have been written for the preparation of traffic signal timing documents for the Maryland State Highway Administration’s Office of Traffic and Safety (OOTS). The guidelines and recommendations pertain to signal timings for new or existing traffic signals on state roadways where signals are maintained by SHA. It is anticipated that the guidelines will also be useful for other traffic signals not noted above.

Objectives of Traffic Signal Timing

The overall objective of signal control is to provide for safe and efficient traffic flow at intersections, along arterials, and throughout roadway networks. Retiming signals is a cost-effective technique to improve traffic flow, reduce fuel consumption, eliminate unnecessary stops and delays, improve safety, and reduce impacts to the environment. Regular timing adjustments can delay the need for major roadway reconstruction.

The functional objective of traffic signal timing is to alternate the right-of-way among various phases in such a way as to:

- Provide for the orderly movement of traffic
- Minimize delay to vehicles and pedestrians
- Reduce the potential for safety conflicts
- Maximize the capacity of each intersection approach

The basic objectives are not always compatible. For example, delay may be minimized by using as few phases as possible and the shortest practical cycle length. However, reducing the potential for crashes requires fewer conflicting movements, which leads to multiple phases and longer cycle lengths. It is necessary to exercise engineering judgment to achieve a feasible balance of objectives.
CHAPTER 2
Traffic Engineering Fundamentals

Interim Edition
2. TRAFFIC ENGINEERING FUNDAMENTALS

A. Volume, Demand, and Capacity

Volume, demand, and capacity all have the same unit of measure, vehicles per hour (vph); however, the applications and measurements are different. For example, if vehicles are counted at a defined location for one hour:

- **Volume** is the number of vehicles counted passing the location in one hour
- **Demand** is equal to the volume plus the number of vehicles that wanted to pass the location, during the one-hour time period but were prevented from doing so due to congestion
- **Capacity** is the maximum volume that could be accommodated by the intersection at the defined location

With respect to manual turning movement counts at signalized intersections:

- During non-congested periods, turning movement counts (volumes) are approximately equal to the demand
- During periods of heavy congestion (i.e., over-capacity conditions), turning movement volume counts (volumes) are approximately equal to the capacity for the respective movement. The demand can be determined by counting upstream on the approach where traffic is unaffected by the intersection. These counts should be performed where queues are not present using either automatic traffic recorders (“tube counts”) or system detectors located at an upstream intersection.

*Peak Hour Volume*

For signal timing applications, peak hour volumes are required for each turning movement. Peak hour volumes are determined by first identifying the peak hour for the intersection, which is the 60-minute time period with the highest total intersection volume for four consecutive 15-minute periods. The corresponding 60-minute volumes for each movement within the peak hour are the peak hour volumes.
Example 2.1: Turning movement counts were performed at a signalized intersection between 7:00 AM and 9:00 AM. The following is the total intersection volume (i.e., sum of all individual turning movement volumes) summarized into 15-minute intervals.

<table>
<thead>
<tr>
<th>Time period</th>
<th>15-minute volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:00 – 7:15 AM</td>
<td>271</td>
</tr>
<tr>
<td>7:15 – 7:30 AM</td>
<td>307</td>
</tr>
<tr>
<td>7:30 – 7:45 AM</td>
<td>358</td>
</tr>
<tr>
<td>7:45 – 8:00 AM</td>
<td>361</td>
</tr>
<tr>
<td>8:00 – 8:15 AM</td>
<td>316</td>
</tr>
<tr>
<td>8:15 – 8:30 AM</td>
<td>319</td>
</tr>
<tr>
<td>8:30 – 8:45 AM</td>
<td>336</td>
</tr>
<tr>
<td>8:45 – 9:00 AM</td>
<td>334</td>
</tr>
</tbody>
</table>

Step 1: Calculate the volume for each 60-minute time period (i.e., four consecutive 15-minute periods).

<table>
<thead>
<tr>
<th>Time period</th>
<th>60-minute volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:00 – 8:00 AM</td>
<td>271 + 307 + 358 + 361 = 1,297</td>
</tr>
<tr>
<td>7:15 – 8:15 AM</td>
<td>307 + 358 + 361 + 316 = 1,342</td>
</tr>
<tr>
<td>7:30 – 8:30 AM</td>
<td>358 + 361 + 316 + 319 = 1,354</td>
</tr>
<tr>
<td>7:45 – 8:45 AM</td>
<td>361 + 316 + 319 + 336 = 1,332</td>
</tr>
<tr>
<td>8:00 – 9:00 AM</td>
<td>316 + 319 + 336 + 334 = 1,305</td>
</tr>
</tbody>
</table>

The peak hour is 7:30 – 8:30 AM and the peak hour volume equals 1,354 vph.

Step 2: Calculate the peak hour volume for each movement using the same four consecutive 15-minute periods as determined in Step 1. For example, the following count data was collected for the westbound through movement:

<table>
<thead>
<tr>
<th>Time period</th>
<th>15-minute volume westbound through</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:00 – 7:15 AM</td>
<td>41</td>
</tr>
<tr>
<td>7:15 – 7:30 AM</td>
<td>70</td>
</tr>
<tr>
<td>7:30 – 7:45 AM</td>
<td>58</td>
</tr>
<tr>
<td>7:45 – 8:00 AM</td>
<td>61</td>
</tr>
<tr>
<td>8:00 – 8:15 AM</td>
<td>63</td>
</tr>
<tr>
<td>8:15 – 8:30 AM</td>
<td>39</td>
</tr>
<tr>
<td>8:30 – 8:45 AM</td>
<td>33</td>
</tr>
<tr>
<td>8:45 – 9:00 AM</td>
<td>43</td>
</tr>
</tbody>
</table>

The peak hour volume for the westbound through movement is 221 vph.
**Peak Hour Factor (PHF)**

The peak hour factor (PHF) is the measure of variation, or fluctuation, in volume during the peak hour. The PHF is defined as follows:

\[
PHF = \frac{\text{peak hour volume}}{4 \times (\text{peak 15-minute volume})}
\]

The PHF can be calculated for each movement, approach, and/or for the total intersection. The PHF ranges between 0.25 and 1.0, and a lower PHF indicates a higher degree of variation in the peak hour volume. The PHF for the intersection in Example 2.1 is calculated as follows:

\[
PHF = \frac{1,354}{4 \times 361} = 0.94
\]

Similarly, the PHF for the westbound through movement in Example 2.1 is calculated as follows:

\[
PHF = \frac{221}{4 \times 63} = 0.88
\]

Although signal timings are typically based on hourly volumes, the PHF can be used to adjust the peak hour volume to account for significant fluctuations in demand (e.g., at intersections that serve schools or businesses with shift work). The adjusted volume is typically referred to as the maximum flow rate, which can be used to increase the amount of green time for the specific movement or approach that experiences the fluctuations.

\[
\text{maximum flow rate} = \frac{\text{peak hour volume}}{PHF}
\]

**Saturation Flow Rate**

The saturation flow rate is the theoretical maximum number of vehicles that can cross the stop line in one approach lane if the approach signal was continuously green. The saturation flow rate depends on roadway geometrics, driver characteristics, and traffic conditions, which can vary substantially for different lanes and approaches at a signalized intersection. For signal timing purposes, lanes or approaches with low saturation flow rates require more green time each cycle to serve the same number of vehicles as lanes or approaches with high saturation flow rates.

The saturation flow rate can be computed using the formula and adjustment factors presented in the *Highway Capacity Manual (HCM)* or measured in the field. The computational HCM method uses a base saturation flow rate of 1,900 vehicles per hour per lane (vphpl), and then the value is adjusted for grade, lane width, intersection location, and adjacent parking. The process for measuring the saturation flow rate in the field is provided on the following worksheet.
# Saturation Flow Study – Field Worksheet

Location: ________________________________________________
Direction: _______________________________ Lane: _______________________________
Date: ___________________ Time: _______________ Observers: ____________________________

## Instructions

1. At the beginning of the green interval, identify the fourth and last vehicles in the queue. The queue must be at least seven vehicles long.
2. If the queue is more than ten vehicles long, identify the tenth vehicle in the queue.
3. Start the stopwatch when the rear axle of the fourth vehicle in the queue crosses the stop line.
4. Stop the stopwatch when the rear axle of the last vehicle in the queue (i.e., the seventh, eighth, ninth, or tenth vehicle) crosses the stop line. For example, if there were nine vehicles in the queue at the start of green, then only the time between the fourth and ninth vehicle should be recorded that cycle.
5. Repeat steps 1 – 4 for the next cycle.

<table>
<thead>
<tr>
<th>CYCLE</th>
<th>TIME (SECONDS) BETWEEN 4\textsuperscript{TH} VEHICLE IN QUEUE AND …</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7\textsuperscript{TH} VEHICLE</td>
</tr>
<tr>
<td>1</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
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<td>5</td>
<td></td>
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<td>17</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
</tr>
</tbody>
</table>

Column Sums:

\[ \text{Measured Saturation Flow Rate (vph)} = \frac{3600 \times \text{total number of cycles observed}}{(a) + (b) + (c) + (d)} \]

\[ = \frac{3600 \times \text{total number of cycles observed}}{3 + 4 + 5 + 6} \]
Capacity

The capacity of a lane at a signalized intersection is determined by factoring the saturation flow rate by the percentage of green time allocated to serve the lane. The capacity is computed by:

\[ c = s \times \frac{g}{C} \]

where,
- \( c \) = capacity of lane, vphpl
- \( s \) = saturation flow rate, vphpl
- \( g \) = green time for the phase serving the lane, sec
- \( C \) = cycle length, sec

**Example 2.2:** The following data was collected at a signalized intersection:

- Cycle length, \( C = 60 \) sec
- East/West through phase green time, \( g = 20 \) sec
- North/South through phase green time, \( g = 30 \) sec
- Yellow plus all-red time (each phase), \( Y+AR = 5 \) sec
- Saturation flow rate (all approaches), \( s = 1,900 \) vphpl

Step 1: Calculate the capacity for the east/west through phase.

\[ c = 1,900 \text{ vphpl} \times \frac{20 \text{ sec}}{60 \text{ sec}} = 633 \text{ vphpl} \]

Step 2: Calculate the capacity for the north/south through phase.

\[ c = 1,900 \text{ vphpl} \times \frac{30 \text{ sec}}{60 \text{ sec}} = 950 \text{ vphpl} \]

Step 3: Calculate the capacity for the entire intersection.

\[ c = 1,900 \text{ vphpl} \times \frac{50 \text{ sec}}{60 \text{ sec}} = 1,583 \text{ vphpl} \]

It is important to understand the relationship between cycle length and capacity. For example, if the cycle length is increased and the number of phases remains the same, the percentage of green time per hour is increased (assuming the yellow plus all-red intervals remain constant) which increases capacity. However, it is important to point out that maximizing capacity is not always the primary goal of signal timing. Effective signal timing balances capacity, safety, queue lengths, and delay.
### V/C Ratio

The V/C ratio measures how the demand at an intersection is served by the capacity. The V/C ratio is typically calculated as follows:

\[
V/C = \frac{\text{Peak Hour Volume}}{\text{Capacity}} = \frac{(\text{Peak Hour Volume}) \times C}{s \times g}
\]

where,
- \( s \) = saturation flow rate, vphpl
- \( g \) = green time for the phase serving the lane, sec
- \( C \) = cycle length, sec

Although the V/C ratio can be calculated for a single lane, lane groups, approaches, or the entire intersection, the most common application is to calculate the V/C ratio for each critical movement (see Critical Lane Volume Analysis). It is desirable that all phases be timed to provide sufficient capacity to handle the demand (i.e., the V/C ratio for each phase should be less than 1.0). A common technique is to distribute the green time so that all phases have the same V/C ratio.

### Over-Capacity Conditions

Intersections that are over capacity (i.e., have a V/C ratio greater than 1.0) are often characterized by excessive delays and long queues that carry over from cycle to cycle or extend into upstream intersections. When queues “spill back” into an upstream intersection, the capacity of the upstream intersection is reduced because vehicles are restricted from entering or clearing the intersection at the saturation flow rate. Similarly, long queues at isolated intersections often block storage lanes or extend into adjacent travel lanes, which reduces the capacity of the blocked movement(s).

The following are common techniques that focus on queue management at over-capacity intersections.

- **Shorter cycle lengths** – As cycle lengths increase, queue and platoon lengths increase. By reducing the cycle length, the likelihood of intersection blockage is decreased.
- **Adjusted offsets** – Instead of using typical time-distance offsets based on travel speed between signals, offsets can be adjusted to allow cross-street traffic to be served at upstream intersections while queues clear at the downstream signal
- **Imbalanced splits** – Rather than allocating the available green in proportion to the V/C ratios, the green time can be proportioned based on available queue storage
Volume Patterns and Timing Plans

In determining signal timing parameters, transportation professionals are often faced with the challenge of variations in traffic volume. Traffic demand can vary by time of day, day of the week, month or season, and during special events (both planned and unplanned) such as construction detours, incidents, and severe weather. In order for a signalized intersection or signal system to operate efficiently, timing plans should be developed for different traffic conditions. Two common types of traffic volume variation are:

- Traffic flow at isolated intersections: Traffic volumes can increase or decrease along one or more approaches at a signalized intersection. The changes can alter the requirements for splits, cycle length, and/or actuation settings.
- Directional traffic flow along arterials: Volume can vary directionally along a two-way arterial used as a commuter route into/out of an urban area or business district as follows:
  1. Inbound volume is greater than outbound volume (usually in the AM peak). The timing pattern should use splits and offsets that favor progression in the inbound direction.
  2. Inbound volume is approximately the same as outbound flow (usually in the off-peak hours and/or mid-day). The timing pattern should equally favor traffic in both directions.
  3. Outbound volume is greater than inbound flow (usually in the PM peak). The timing pattern should use splits and offsets that favor progression in the outbound direction.

Signal timing patterns consist of unique combinations of cycle length, splits, and offsets. Volume trends can be identified by performing volume studies on various approaches (e.g., turning movement counts and ATR counts). Typically, patterns are developed for the following periods:

- AM peak
- Mid-day
- PM peak
- Night
- Weekend
- Special (e.g., evacuation plans, schools, shift work, sporting events, etc.)

B. Performance Measures

Performance measures allow transportation professionals to evaluate signal timing modifications for various timing objectives and operational goals. Common measures associated with signal timing include travel time, delay, and queue length. Safety experience (i.e., crash history) is also considered a performance measure for signal timing.

Travel Time and Delay

In general terms, travel time is the total time required to travel a given distance, such as the length of a signalized arterial, including the delay at intersections and the running time, which is the time required to travel a given distance at a given speed. The delay at intersections, often referred to as control delay, is the time required to decelerate to and accelerate from a stop as well as the time a
vehicle spends stopped waiting to proceed through the signal. Figure 2.1 illustrates the relationship between travel time and delay using a time-distance diagram.

Performing travel time and delay studies at isolated intersections or along coordinated signal systems enables transportation professionals to identify problem areas in vehicle progression, number of stops, and control delay.

Additional information and procedures for conducting travel time and delay studies can be found in ITE’s Manual of Transportation Studies (2000 edition, Chapter 4).

Queue Length

Queue length is a critical measurement for traffic signal efficiency and intersection design. Specific signal timing schemes can be developed to focus on managing queues, which may cause an increase in overall intersection delay. For example, at over-capacity intersections, timings are often modified to prevent queues from exceeding or blocking storage lanes or extending into upstream intersections. Common queuing-related problems, which may require modifications to signal timing parameters, include:

- Vehicles in the queue at the beginning of green do not clear the intersection within one cycle length
- Queues exceed turn lane storage bays and block adjacent through lanes
- Through queues restrict access to adjacent turn lanes
- Queues extend into upstream intersections and restrict vehicles from entering the intersection during the green interval and/or block cross-street movements
Typically, queue length studies are conducted by counting the number of vehicles stopped or slowly moving in a queue during designated time intervals, such as the start of the green interval and the end of the yellow interval. Figure 2.2 provides a graphical representation of the queue length at a signalized intersection.

![Figure 2.2 – Queue length at a signalized intersection](image)

As shown in Figure 2.2, vehicles cannot depart from the intersection during the red interval and a queue is formed. Queue lengths can be measured in feet or number of vehicles.

Additional information and procedures for conducting queuing studies can be found in ITE’s *Manual of Transportation Studies* (2000 edition, Chapter 5).

**C. Critical Lane Volume Analysis**

The critical lane volume (CLV) analysis focuses on “raw” intersection capacity, that is, the ability for an intersection to serve demand for given lane configurations and signal phasing. The CLV analysis is a fundamental tool for calculating green times and evaluating signal phasing schemes because it identifies movements that are “critical” to the signal operations. Specifically, the results provide a baseline for determining signal timing parameters such as splits and cycle length.

According to ITE’s *Traffic Signal Timing Manual*, “The amount of time in an hour is fixed, as is the fact that two vehicles (or a vehicle and a pedestrian) cannot safely occupy the same space at the same time. Critical [lane volume] analysis identifies the set of movements that cannot time concurrently and require the most time to serve demand.” Additionally, CLV analysis incorporates the following basic assumptions for intersection geometrics and traffic flow:
○ Lane widths and grades on the intersection approaches are “typical.” No adjustments are made for specific widths or grades.
○ Adjustments are not made for the specific composition of traffic. The proportion of trucks, motorcycles, bicycles, and buses do not affect signal operations.
○ Pedestrians do not conflict with turning vehicles
○ Right-turning and left-turning vehicles discharge through the intersection at the same rate as through vehicles
○ Traffic does not equally distribute amongst multiple lanes on an approach

The examples contained in Chapters 4, 6, and 9 of this manual are primarily based on the CLV methodology. Additional information regarding SHA’s specific CLV procedures and guidelines can be found in Chapter 11, the State Highway Access Manual (Appendix E), or the Highway Capacity Manual (Chapter 10, Appendix A).
CHAPTER 3
Data Collection

Interim Edition
3. DATA COLLECTION

Data collection is an important element of the signal timing process. All data should be recent enough to reflect current conditions. Data collection should include, but is not limited to:

- **Intersection geometry and field measurements**
  - lane usage (left/through/right)
  - turn lane storage lengths
  - distances between intersections
  - “curb-to-curb” crosswalk distances
  - posted speed limits
  - right-turn control (e.g., Yield, Stop, and “free”)
  - acceleration lanes/merge lanes

- **Volume counts**
  - turning movement counts for each study period (e.g., weekday AM, PM, and mid-day peaks; weekend and “off peak” counts, when appropriate)
  - main street through movement counts (e.g., seven day counts using system masters and automatic traffic recorders (ATRs))
  - pedestrian counts
  - heavy vehicle percentages, by class

- **Existing signal equipment and operations**
  - signal plan
  - preemption receivers, if present
  - detector types, locations, and phase assignments
  - phase sequences and overlaps
  - existing timing charts

- **Performance measures and studies – typically used for Synchro calibration**
  - travel time/delay runs
  - approach queue and delay studies
  - saturation flow rates

- **Other information**
  - future signals
  - future roadway improvements/geometric modifications
  - planned developments

Although on-site measurements and observations often provide a better understanding of intersection characteristics and operations, several resources are available for virtual “site” visits.

- **Google Earth** ([http://earth.google.com](http://earth.google.com)) or **Bing Local** ([http://www.bing.com/local](http://www.bing.com/local)) – Public internet sites with aerial photography that can be used to identify lane usage and perform “field” measurements

- **SHA Video Log** – Site available on the SHA Intranet that can be used to identify intersection geometry and lane configurations

- **SHA Synchro Library** – Database of existing signal operations and turning movement counts. Available on the SHA Intranet.
- **SHA Traffic Monitoring System** ([http://www.sha.state.md.us/tmsreports](http://www.sha.state.md.us/tmsreports)) – Public internet site with database of ATR and turning movement counts


- **SHA Electronic File Room** – Database of existing roadway plans maintained by the Office of Highway Development, Highway Design Division
CHAPTER 4
Signal Phasing

Interim Edition
4. SIGNAL PHASING

A *phase* is the assignment of right-of-way (i.e., green plus yellow intervals) to a movement or combination of movements. The order in which all phases at an intersection are displayed during the cycle is the *phase sequence*. The objectives of signal phasing are to minimize conflicting movements and to maintain efficient traffic flow through the intersection by reducing delays and queues.

The timing procedures presented in this chapter for various phasing types represent the initial approach and basic methodology for determining green times based on critical lane volumes. The techniques for calculating specific green intervals and splits are provided in Chapters 6 and 9.

A. **NEMA Phase Numbers**

The number of phases at an intersection is based upon left-turn treatments, intersection geometry, lane usage, and volumes (both vehicular and pedestrian). Typically, two to eight phases are assigned; however, additional phases can be used at complex intersections.

- Phase numbers typically are based upon the National Electrical Manufacturers Association (NEMA) phasing structure.

Figures 4.1a and 4.1b show typical NEMA phase numbers for an intersection with an east-west main street and north-south main street, respectively.

As shown in the figures, phase 1 is typically assigned to the eastbound left-turn movement at an intersection with an east-west main street or it is usually assigned to the northbound left-turn movement at an intersection with a north-south main street.
Figure 4.2 illustrates the typical NEMA phase sequence for an intersection with eight phases and an east-west main street.

![Figure 4.2 – Traditional NEMA phase sequence (8 phases)](image)

Additional information on NEMA phasing can be found in the NEMA Standards Publication TS 2-2003: Traffic Controller Assemblies with NTCIP Requirements and the Federal Highway Administration’s Signalized Intersections: Informational Guide.

B. **Left-Turn Phases**

Determining signal phasing typically begins with selecting the appropriate left-turn phases, which requires balancing safety and efficiency. There are three basic types of left-turn phasing: permissive, exclusive/permissive, and exclusive. The selection of a specific type of left-turn phasing is typically part of the signal design process, which includes an evaluation of SHA’s left-turn phasing guidelines found in SHA’s Traffic Control Device Application Guidelines Manual.

![Figure 4.3 – Balance between safety and efficiency for left-turn phasing](image)
Permissive (Permitted) Left-Turn Phase

With permissive left-turn phasing, left-turning vehicles are allowed to turn on a circular green indication (“green ball”) after yielding to opposing traffic.

Timing permissive left-turn phases

Because left-turning vehicles yield to opposing through vehicles, the total green time for each through phase should be long enough to service both the through volume and opposing left-turn volume.

\[
\text{Critical lane volume for through phase with opposing permissive left turns} = \text{Lane volume for through movement} + \text{Lane volume for opposing left-turn movement}
\]
Example 4.1: Determine the critical lane volume for each through phase:

- Permissive left-turn phasing on all approaches
- Lane use factor (LUF) for two through lanes: 0.55 (see Chapter 11)

Step 1: Calculate the equivalent lane volume for the northbound and southbound through movements.

\[
\text{NB through lane volume} = 300 \text{ vph} \times 0.55 = 165 \text{ vph} \\
\text{SB through lane volume} = 200 \text{ vph} \times 0.55 = 110 \text{ vph}
\]

Step 2: Add the opposing left-turn lane volumes to the through lane volumes to calculate the critical lane volume for the through phases.

\[
\begin{align*}
\text{NB through phase CLV} &= 165 + 65 = 230 \text{ vph} \\
\text{SB through phase CLV} &= 110 + 80 = 190 \text{ vph} \\
\text{EB through phase CLV} &= 75 + 30 = 105 \text{ vph} \\
\text{WB through phase CLV} &= 140 + 115 = 255 \text{ vph}
\end{align*}
\]
Exclusive/Permissive (Protected/Permitted) Left-Turn Phase

Left-turning vehicles are allowed to turn either on a green arrow or on a circular green indication after yielding to opposing traffic. These vehicles have the right-of-way only while the green arrow is displayed.

Figure 4.5 – Phase sequence for exclusive/permissive left-turn phasing

Timing exclusive/permissive left-turn phases

Timing exclusive/permissive left-turn phases requires two separate calculations – the exclusive portion (i.e., left-turn phase) and the permissive portion (i.e., the opposing through phase).

The time required for the exclusive portion is based upon the left-turn lane volume.

Timing the permissive portion depends upon the sum of the left-turn lane volume and the opposing through lane volume because the time must be long enough to accommodate both movements.

Critical lane volume for the exclusive portion (i.e., left-turn phase)

Lane volume for the left-turn movement

Critical lane volume for the permissive portion (i.e., opposing through phase)

Lane volume for the left-turn movement + Lane volume for the opposing through movement
Example 4.2: Determine the critical lane volume for the exclusive and permissive portions for each exclusive/permissive phase:

- Exclusive/permissive left-turn phasing on all approaches
- Lane use factor (LUF) for two through lanes: 0.55 (see Chapter 11)

Step 1: Determine the critical lane volume for the exclusive portions.

- NB exclusive left - turn phase \( CLV = 200 \text{ vph} \)
- SB exclusive left - turn phase \( CLV = 135 \text{ vph} \)
- EB exclusive left - turn phase \( CLV = 215 \text{ vph} \)
- WB exclusive left - turn phase \( CLV = 150 \text{ vph} \)

Step 2: Calculate the equivalent lane volumes for the northbound and southbound through movements.

- NB through lane volume = \( 500 \text{ vph} \times 0.55 = 275 \text{ vph} \)
- SB through lane volume = \( 400 \text{ vph} \times 0.55 = 220 \text{ vph} \)

Step 3: Add the opposing left-turn volumes (from Step 1) to the through movement lane volumes to calculate the critical lane volume for the through phases.

- NB through phase \( CLV = 275 + 135 = 410 \text{ vph} \)
- SB through phase \( CLV = 220 + 200 = 420 \text{ vph} \)
- EB through phase \( CLV = 175 + 150 = 325 \text{ vph} \)
- WB through phase \( CLV = 300 + 215 = 515 \text{ vph} \)
**Flashing Red Arrow Left-Turn Phase**

The flashing red arrow left-turn phase is a type of exclusive/permissive phasing. It is considered an intermediate “step” between exclusive/permissive and strictly exclusive phasing because it can reduce the number of conflicts between left-turning and opposing through vehicles. SHA occasionally uses variable left-turn phasing with a phase operating exclusive only during peak periods when conflicting volumes are highest and then operating with a flashing red arrow phase during off-peak periods to reduce left-turn queues and delays.

Left-turning vehicles are allowed to turn either on a green arrow or on a flashing red arrow after stopping and yielding to opposing traffic. These vehicles have the right-of-way only while the green arrow is displayed.

First, the green arrow indication is displayed and then the yellow arrow indication. Next, a solid red arrow is displayed, followed by the flashing red arrow indication. The flashing red arrow is “delayed” (i.e., the time when the solid arrow is displayed) to allow the queue of opposing through vehicles to be served prior to allowing the permissive left turns portion of the movement.

![Figure 4.6 – Phase sequence for flashing red arrow left-turn phasing](image)

**Timing flashing red arrow left-turn phases**

The procedures for timing flashing red arrow left-turn phases are similar to exclusive/permissive timing – i.e., two separate calculations for the exclusive and permissive portions. However, the “delay” for the flashing red arrow, which depends upon the opposing through volume, must also be considered. Typically, the delay is equal to the minimum green time for the opposing through movement (see Chapter 6).

\[
\begin{align*}
\text{Critical lane volume for the exclusive portion (i.e., left-turn phase)} & = & \text{Lane volume for the left-turn movement} \\
"\text{Delay" for solid red arrow indication before flashing red arrow indication} & = & \text{Minimum green time for the opposing through phase (typical) or time required to serve queued opposing through vehicles} \\
\text{Critical lane volume for the permissive portion (i.e., opposing through phase)} & = & \text{Lane volume for the left-turn movement} + \text{Lane volume for the opposing through movement}
\end{align*}
\]
Exclusive (Protected) Left-Turn Phase

Left-turning vehicles are allowed to turn only on a green arrow indication.

![Figure 4.7 – Phase sequence for exclusive left-turn phasing]

**Timing exclusive left-turn phases**

Because left-turning vehicles are served separately from opposing through vehicles, the total green time for exclusive left-turn phases is based upon the critical lane volume for the left-turn movement.
Example 4.3: Determine the critical lane volume for each exclusive left-turn phase:

- Exclusive left-turn phasing on all approaches
- Lane use factor (LUF) for double left-turn lanes: 0.60 (see Chapter 11)

Step 1: Calculate the equivalent lane volumes for the northbound and southbound left-turn movements.

\[
\begin{align*}
\text{NB left-turn lane volume} &= 450 \text{ vph} \times 0.60 = 270 \text{ vph} \\
\text{SB left-turn lane volume} &= 350 \text{ vph} \times 0.60 = 210 \text{ vph}
\end{align*}
\]

Step 2: Identify the critical lane volume for each left-turn phase.

\[
\begin{align*}
\text{NB left-turn phase CLV} &= 270 \text{ vph} \\
\text{SB left-turn phase CLV} &= 210 \text{ vph} \\
\text{EB left-turn phase CLV} &= 215 \text{ vph} \\
\text{WB left-turn phase CLV} &= 150 \text{ vph}
\end{align*}
\]

C. Phase Sequences

Split (Separate) Phasing

When two opposing side street approaches operate separately, rather than simultaneously, the phasing is referred to as split phasing.

Split phasing may be implemented if an approach has a shared left-turn/through lane (often as a result of relatively high left-turn volumes that require double left-turn lanes on a three-lane approach) or if the alignment of the side street approaches does not physically allow simultaneous movements or causes confusion or operational problems (e.g., interlocking turns). Split phasing also is implemented if safety problems exist with concurrent operations. Referring to Figure 4.3, split...
phasing typically is considered the “safest” left-turn treatment but it also can be the most inefficient. As a result, split phasing should only be implemented after an evaluation of the side street traffic volumes and lane configurations indicates that split phasing is the most appropriate operation.

Phase numbers for split phasing do not follow the traditional NEMA phasing structure. As shown in Figure 4.8, the minor street phases typically are numbered Phase 3 and Phase 4 because standard controllers do not allow these phases to operate concurrently.

![Figure 4.8 – Typical phase sequence for split phasing on the minor street](image)

If the approach volumes on the minor street are imbalanced, then the approach with the higher volume typically is sequenced after the lower-volume approach and it is assigned to Phase 4. This allows “excess” time from Phase 3 (i.e., when Phase 3 terminates early) to be reallocated to Phase 4.

Additionally, if an intersection with split phasing is part of a coordinated system, then the phase sequence can be designed to provide coordination for a minor street movement at a downstream intersection. For example, if the northbound left-turn movement, denoted as Phase 3 in Figure 4.8, can be “platooned,” or progressed, with the westbound movement because it is sequenced immediately after Phase 6.

### Timing split phases

Because the opposing approaches are “split,” the green time required to serve each approach is based on the critical lane volume for each approach.

\[
\text{Critical lane volume for each approach} = \max\left(\text{Right-turn lane volume, Left-turn lane volume, Through lane volume, Shared lane volume}\right)
\]
**Example 4.4:** Determine the critical lane volume for each approach (split) phase:

**Split phasing on the northbound and southbound approaches**

- Lane use factor (LUF) for two through lanes: 0.55 (see Chapter 11)
- Lane use factor (LUF) for double left-turn lanes: 0.60 (see Chapter 11)

**Step 1:** Calculate the equivalent lane volumes for the northbound and southbound movements.

<table>
<thead>
<tr>
<th>Movement</th>
<th>Equivalent Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>NB left - turn</td>
<td>400 vph × 0.60 = 240 vph</td>
</tr>
<tr>
<td>NB through lane</td>
<td>500 vph × 0.55 = 275 vph</td>
</tr>
<tr>
<td>SB left - turn</td>
<td>350 vph × 0.60 = 210 vph</td>
</tr>
<tr>
<td>SB through lane</td>
<td>150 vph</td>
</tr>
</tbody>
</table>

**Step 2:** Identify the critical lane volume for each phase.

<table>
<thead>
<tr>
<th>Phase</th>
<th>CLV</th>
</tr>
</thead>
<tbody>
<tr>
<td>NB split</td>
<td>max(240 or 275 vph) = 275 vph</td>
</tr>
<tr>
<td>SB split</td>
<td>max(210 or 150 vph) = 210 vph</td>
</tr>
</tbody>
</table>

**Concurrent (Simultaneous) Phasing**

When two non-conflicting left-turn movements operate at the same time, or simultaneously, the phasing is referred to as *concurrent phasing*. To provide through-traffic progression in a coordinated system, it may be beneficial to operate some intersections with lead left-turn phases and some with lead/lag left-turn phases. More information on signal system timing is provided in Chapter 9.
**Lead left-turn phase (one direction)**

An exclusive left-turn phase operates concurrently with the through phase in the same direction. The phase is sequenced before the opposing through phase.

![Figure 4.9a - Phase sequence for exclusive/permissive lead left-turn phasing](image)

**Dual lead left-turn phases (both directions)**

Exclusive left-turn phases, on opposing approaches, are served concurrently and sequenced before the through phases.

![Figure 4.10a - Phase sequence for exclusive/permissive dual lead left-turn phasing](image)

**Lag left-turn phase (one direction only)**

An exclusive left-turn phase operates concurrently with the through phase in the same direction. The phase is sequenced after the opposing through phase. Lag left-turn phasing is SHA’s preferred sequence at “T” intersections with exclusive/permissive left-turn phasing (see Figure 4.11a), because lag phasing allows left-turning vehicles to be served during the initial permissive portion, which can eliminate the need for the exclusive portion of the movement. However, if lead left-turn phasing improves system coordination or queue storage, then lead left-turn phasing should be implemented instead of lag phasing.

![Figure 4.10b - Phase sequence for exclusive dual lead left-turn phasing](image)
Lead/lag left-turn phases

First, an exclusive lead left-turn phase operates in one direction. Next, the through movements are served, and then an exclusive lag left-turn phase operates in the opposite direction. Lead/lag left-turn phasing is occasionally used along coordinated signal systems to provide progression in two directions.
Conditional left-turn phases (phase re-servicing or lead plus lag)

Although infrequently used, a lead left-turn phase can be serviced twice per cycle, i.e., operated again as a lag phase, when the opposing through phase terminates early ("gaps out") and the through phase in the same direction is still being serviced.

Conditional left-turn phases can be used when inadequate left-turn storage is available and/or if platoons arrive after the exclusive lead left-turn phase terminates.

**Figure 4.13a – Phase sequence for exclusive/permissive phase re-servicing**

**Figure 4.13b – Phase sequence for exclusive phase re-servicing**

Yellow trap (left-turn trap)

The combination of a permissive left-turn phase and an opposing lag phase can lead to a situation commonly referred to as the "yellow trap." As shown in Figures 4.14a and 4.14b (which correspond to Figures 4.11a and 4.12a), permissive left turns typically are prohibited in the opposing direction of a lag phase.

**Figure 4.14a – Yellow trap with exclusive/permissive lag left-turn phasing (one direction)**

**Figure 4.14b – Yellow trap with exclusive lead/lag left-turn phasing**
If these scenarios are permitted, a left-turning vehicle, present at the end of the through phase (i.e., still awaiting a gap when the “yellow ball” is displayed), may incorrectly presume that the opposing through phase also is ending. When the signal turns red, the left-turning vehicle may get “trapped” in the intersection or attempt to complete the turn into oncoming traffic.

D. Right-Turn Phases

Exclusive Right-Turn Phase

Although the use of exclusive left-turn phasing at intersections is common, the majority of right-turn movements operate without exclusive signal phasing.

Exclusive right-turn phases typically are used for special conditions, such as:
- when there is a need to separate right-turning traffic from high pedestrian traffic volumes
- where more than one right-turn lane exists on an approach
- where right-turning vehicles have a separate lane and can operate concurrently with left-turn phases on the cross street (see concurrent and overlap right-turn phases)
- on minor streets along coordinated signal systems to ensure more “predictable” platoon arrivals and improve progression (infrequently used)

Right-turn on red prohibitions along an approach or for specific lanes (e.g., the leftmost right-turn lane on an approach with double right-turn lanes) need to be considered when timing exclusive right-turn phases or phases with signal-controlled right-turn movements. These turn restrictions may cause the right-turn lane volume to be “critical” for the signal phase, and, as a result, adequate green time should be provided to serve the right-turn volume.

Timing exclusive right-turn phases

Exclusive right-turn phases typically operate simultaneously with higher volume movements on the same approach (i.e., left-turn or through phases). However, if the right-turn volume is the critical lane volume for the approach, then the total green time for the approach is based upon the right-turn critical lane volume.
Example 4.5: Determine the critical lane volume for the westbound phase:

- Westbound phase has an exclusive right-turn phase.
- Exclusive pedestrian phase
- Lane use factor (LUF) for double right-turn lanes: 0.55 (see Chapter 11)

Step 1: Calculate the equivalent lane volume for the westbound right-turn movement.

\[
\text{WB right - turn lane volume} = 800 \text{ vph} \times 0.55 = 440 \text{ vph}
\]

Step 2: Identify the critical lane volume for the westbound phase.

\[
\text{WB phase CLV} = \max(440 \text{ or } 350 \text{ vph}) = 440 \text{ vph}
\]
**Concurrent Right-Turn Phase**

Exclusive right-turn phases occasionally are operated concurrently with the exclusive left-turn phase on the cross street to increase capacity and serve right-turning vehicles without pedestrian conflicts.

![Image of concurrent right-turn phasing](image.png)

**Figure 4.15 – Phase sequence for concurrent right-turn phasing**

### Timing concurrent right-turn phases

Because a portion of the right-turn volume is served concurrently with the cross street left-turn phase, a credit is applied to the right-turn critical lane volume. The credit is equal to the concurrent left-turn lane volume. If the “credited” right-turn volume is the critical lane volume for the approach, then the total green time for the approach is based on the right-turn critical lane volume.

\[
\text{Credit for right-turn phase critical lane volume} = \text{Lane volume for concurrent cross street left-turn phase}
\]
Example 4.6: Determine the critical lane volume for the westbound phase:

- Westbound right-turn phase operates concurrently with southbound left-turn phase
- Concurrent pedestrian phase
- Lane use factor (LUF) for double left-turn lanes: 0.60 (see Chapter 11)

Step 1: Calculate the credit for the right-turn lane volume based on the southbound left-turn lane volume.

\[ \text{WB right - turn credit} = \text{SB left - turn lane volume} = 350 \text{ vph} \times 0.60 = 210 \text{ vph} \]

Step 2: Calculate the right-turn critical lane volume.

\[ \text{WB right - turn CLV} = 530 - 210 \text{ vph} = 320 \text{ vph} \]

Step 3: Identify the critical lane volume for the westbound phase.

\[ \text{WB phase CLV} = \max(320 \text{ or } 200 \text{ vph}) = 320 \text{ vph} \]
Overlap Right-Turn Phase

Overlap right-turn phases are similar to concurrent right-turn phases because the right-turn movement operates with the exclusive left-turn phase on the cross street. The difference is that during the approach phase for right-turn lane, the right-turn phase is exclusive, rather than permissive.

![Phase sequence for overlap right-turn phasing](image)

Figure 4.16 – Phase sequence for overlap right-turn phasing

**Timing overlap right-turn phases**

Because a portion of the right-turn volume is served concurrently with the cross street left-turn phase, a credit is applied to the right-turn critical lane volume. The credit is equal to the concurrent left-turn lane volume. If the “credited” right-turn volume is the critical lane volume for the approach, then the total green time for the approach is based upon the right-turn critical lane volume.

\[
\begin{align*}
\text{Credit for right-turn phase} & = \text{Lane volume for concurrent cross street left-turn phase} \\
\text{critical lane volume} & \\
\end{align*}
\]
**Example 4.7:** Determine the critical lane volume for the westbound phase:

1. Westbound right-turn phase operates concurrently with southbound left-turn phase
2. Exclusive pedestrian phase
3. Lane use factor (LUF) for double left-turn lanes: 0.60 (see Chapter 11)
4. Lane use factor (LUF) for double right-turn lanes: 0.55 (see Chapter 11)

**Step 1:** Calculate the equivalent lane volume for the westbound right-turn movement.

\[
\text{WB right - turn lane volume} = 800 \text{ vph} \times 0.55 = 440 \text{ vph}
\]

**Step 2:** Calculate the credit for the right-turn lane volume based on the southbound left-turn lane volume.

\[
\text{WB right - turn credit} = \text{SB left - turn lane volume} = 350 \text{ vph} \times 0.60 = 210 \text{ vph}
\]

**Step 3:** Calculate the right-turn critical lane volume.

\[
\text{WB right - turn CLV} = 440 - 210 \text{ vph} = 230 \text{ vph}
\]

**Step 4:** Identify the critical lane volume for the westbound phase.

\[
\text{WB phase CLV} = \max(230 \text{ or } 200 \text{ vph}) = 230 \text{ vph}
\]
E. Pedestrian Phases

Determining the appropriate pedestrian phasing requires balancing safety and efficiency. There are three basic types of pedestrian phases: concurrent, advanced (leading), and exclusive.

- **INCREASE IN SAFETY**
  - Reduction in the number of conflicting

- **CONCURRENT**

- **ADVANCED (LEADING)**

- **EXCLUSIVE**
  - Reduction in delays (for both vehicles and pedestrians)

**Figure 4.17 – Balance between safety and efficiency for pedestrian phasing**

**Concurrent Pedestrian Phase**

The pedestrian phase operates concurrently with a non-conflicting (i.e., parallel) vehicle phase, as shown in Figures 4.18a and 4.18b. Right-turning vehicles must yield to pedestrians in the crosswalk before completing the turn. It is important to point out that pedestrian phases shall not operate concurrently with conflicting exclusive left or right-turn phases.

**Figure 4.18a – Phase sequence for concurrent pedestrian phasing (concurrent minor street phasing)**

**Figure 4.18b – Phase sequence for concurrent pedestrian phasing (split minor street phasing)**
Timing concurrent pedestrian phases

Concurrent pedestrian phases consist of two intervals – the walk and flashing don’t walk (clearance) intervals. The walk interval has to be long enough for a small group of pedestrians to begin the crossing maneuver prior to the pedestrian clearance interval. Although a 7-second walk interval is preferred by SHA, engineering judgment should be used to determine if pedestrian volumes and characteristics justify shortening the walk interval to a minimum of 4 seconds or increasing the walk interval for additional “startup” time. The flashing don’t walk (clearance) interval must be long enough to allow a pedestrian to walk from curb line to curb line.

The following is the recommended practice for calculating concurrent pedestrian intervals.

\[
\text{Walk interval} = \begin{cases} 
7 \text{ seconds} \\
\text{(no calculations required)}
\end{cases}
\]

\[
\text{Flashing don’t walk (clearance) interval} = \frac{\text{“Curb-to-curb” crosswalk distance}}{3.5 \text{ ft/sec (walking speed)}}
\]
Example 4.8: Determine the pedestrian walk and flashing don’t walk intervals for the east-west pedestrian phase:

- East-west pedestrian phase operates concurrently with westbound phase

Step 1: Identify the time required for the walk interval.

Walk interval = 7 seconds

Step 2: Calculate the time for the flashing don’t walk interval.

\[
\text{Flashing don’t walk interval} = \frac{63 \text{ feet}}{3.5 \text{ ft/sec}} = 18 \text{ seconds}
\]

Easy technique to check pedestrian intervals in the field

Time the walk interval to ensure it is 7 seconds.

At the end of the walk interval/beginning of flashing don’t walk interval, start walking across the crosswalk – begin at the edge of curb. If the steady don’t walk indication is illuminated before you reach the other side, the flashing don’t walk interval needs to be increased.
**Concurrent pedestrian phases and split phasing**

As shown in Figures 4.18b and 4.19a, to avoid conflicts between left-turning vehicles (exclusive phase) and pedestrians, pedestrian phases on both legs typically are not operated simultaneously with split phasing.

If simultaneous crossings are warranted, the left-turn phase must be permissive (i.e., green ball indication instead of green arrow indication), as shown in Figure 4.19b. However, pedestrian versus permissive left-turning vehicle conflicts generally are not preferred, because the phasing violates typical driver “expectations” – i.e., pedestrians crossing on the right.

![Figure 4.19a – Phase sequence for concurrent pedestrian phasing (split phasing on the minor street - exclusive)](image1)

![Figure 4.19b – Phase sequence for concurrent pedestrian phasing (split phasing on the minor street - permissive)](image2)

**Advanced Pedestrian Phase (Leading Pedestrian Interval)**

To minimize conflicts between pedestrians and vehicles, concurrent pedestrian phases can be timed to begin the walk interval before the start of green for the concurrent vehicle phase or while right turns are temporarily prohibited. The following describes the two different types of advanced pedestrian phases, which provide pedestrians with “lead time” to establish the right-of-way and increase their visibility to conflicting vehicles.
Extended all-red advanced pedestrian phase
The pedestrian phase begins during an extended all-red vehicle clearance interval for conflicting vehicle phases. Typically, the advanced walk interval is 4 to 7 seconds (in addition to the standard 7-second walk interval), which begins at the end of the “original” clearance interval.

Prohibited (temporarily) right turns advanced pedestrian phase
The concurrent pedestrian and vehicle phases begin at the same time. However, a red arrow is displayed to conflicting right-turn movement for 4 to 7 seconds (typical). Next, a green ball is displayed to the right-turn movement (or the red arrow is removed), which requires right-turning vehicles to yield to pedestrians in the crosswalk before completing the turn.

Exclusive Pedestrian Phase
At locations where high volumes of pedestrians conflict with traffic on several approaches, an exclusive pedestrian phase may be suitable. The pedestrian phase is served while all of the approaches receive red indications. Right turns on red typically are prohibited at intersections with an exclusive pedestrian phase.
**Timing exclusive pedestrian phases**

Signal timing for exclusive pedestrian phases consists of three intervals – the walk, flashing don’t walk (clearance), and steady don’t walk (additional clearance) intervals. The steady don’t walk (additional clearance) interval essentially serves as a short all-red interval before the next vehicle phase to minimize pedestrian and vehicle conflicts.

The following is the recommended practice for calculating intervals for exclusive pedestrian phases.

\[
\text{Walk interval} = \begin{cases} 
7 \text{ seconds} \\
\text{(no calculations required)} 
\end{cases}
\]

\[
\text{Flashing don’t walk (clearance) interval} = \frac{\text{Longest “curb-to-curb” crosswalk distance (multiple crossings)}}{3.5 \text{ ft/sec (walking speed)}}
\]

\[
\text{Steady don’t walk interval before green indication for next vehicle phase} = \begin{cases} 
3 - 5 \text{ seconds} \\
\text{(no calculation required)} 
\end{cases}
\]

**Recommendation for Crossings with Wide Medians**

At intersections with wide medians without a median push button, SHA’s preferred duration for the pedestrian clearance interval should be long enough so that pedestrians can cross the entire intersecting roadway on one cycle. Although medians can serve as a “refuge” for pedestrians, requiring two or more cycles for pedestrians to cross could result in a high number of illegal crossings.

At intersections where a push button is provided in the median (i.e., a median “refuge”), the pedestrian clearance interval should be equal to the crossing time (at 3.5 feet per second) required for the longer of the two distances from the curb line to the median, as shown in Figure 4.22.
Pedestrian walk interval (WALK) = 7 seconds (no calculation required)

Pedestrian clearance interval (PED CLEAR) =

Higher of: \( \frac{X_1}{3.5 \text{ ft/sec}} \) or \( \frac{X_2}{3.5 \text{ ft/sec}} \)

Figure 4.22 – Pedestrian intervals for medians with push buttons
CHAPTER 5

Actuated Control

Interim Edition
5. ACTUATED CONTROL

A. Basic Types of Signal Control

There are two basic types of traffic signal control – pre-timed and traffic-actuated. Pre-timed signals operate on a fixed cycle length with preset green intervals for each phase. The phase sequence is also constant, meaning all phases get “called” each cycle. Pre-timed control tends to be used at intersections where traffic is more "predictable," and where fixed cycle lengths, green intervals, and phases are required for coordination, such as in a dense, urban area.

Traffic-actuated signals have varying green intervals for phases equipped with detectors. Traffic-actuated signals do not have a fixed cycle length unless the intersection is in a coordinated system. The phasing sequence also may change from cycle-to-cycle, depending upon the presence of traffic. Actuated control is traditionally used at intersections with varying traffic volumes. Semi-actuated signals typically only have detectors on the minor street approaches and in major street left-turn lanes. Fully-actuated signals have detectors on all approaches.

For both types of control, the yellow change and red clearance intervals for each phase are preset and different timing patterns can be developed for various times of day. Additionally, both types of control can be used at isolated (independent) intersections or at intersections within a signal system.

B. Primary Detection Functions

Signal detection takes on four basic operational functions – passage, presence, queue, and system (sampling).

Passage Detection (Point or pulse detection)

Passage detection indicates that a vehicle has passed through the detection zone. These types of detectors are typically used in through lanes along major street approaches and along high-speed minor street approaches. Passage detection allows the controller to determine the amount of green time required to serve vehicles that have arrived during the red interval, as well as determine the amount of time to extend the green interval when vehicles pass through the detection zone during the green interval.

Although various technologies are available for passage detection, SHA’s preferred type of passage detector is non-invasive microloop probes. In areas where right-of-way, geometrics, or underground utilities restrict the use of the non-invasive probes, intrusive microloop probes, 6-foot by 6-foot inductive loop detectors, or video imaging cameras should be used for passage detection.
Presence Detection

Presence detection indicates that a vehicle is within the detection zone. These detectors are typically installed at the stop line along minor street approaches and in major street left-turn lanes, where it is critical to know if a vehicle is waiting for green.

SHA’s preferred type of presence detector is video imaging; however, 6-foot by 30-foot inductive loops are also commonly used, especially at signals with span wires.

Queue Detection

Queue detectors typically are placed along freeway off-ramps upstream of signalized intersections to detect the presence of queued vehicles and to place a call on the signal controller to modify signal timings to prevent queues from extending back onto the freeway (i.e., preempt the ramp phase or adjust the maximum splits using the split demand function). Queue detectors are occasionally installed at the entrance to left-turn lanes where the left-turn phase may need to be re-serviced during the cycle (i.e., a conditional left-turn phase) to prevent queues from exceeding the storage length.

System Detection (Sampling detection)

System, or sampling, detectors typically are placed downstream of intersections along signalized corridors to collect free flow traffic data, such as volume, speed, and occupancy (density). This information can be collected “real time” to assess the operations and performance of signal systems and allow transportation officials to make timing adjustments for changes in traffic conditions.

SHA’s preferred type of system detector is a 6-foot by 6-foot inductive loop detector.

C. Types of Detectors

The following is a brief introduction of the various types of signal detectors commonly used by SHA. Additional information regarding signal detection, including design considerations and hardware requirements, can be found in SHA’s Traffic Control Devices Design Manual – http://www.marylandroads.com/Index.aspx?PagId=45.

Video Imaging

Video imaging (camera) detectors are SHA’s preferred type of presence detection. However, video imaging can also be used for passage detection.

Video cameras can be installed on signal mast arms, lighting bracket arms, or attached to the upright signal pole; however, signal pole mounted cameras are not preferred if other options are available. The camera location and mounting height depend upon the geometry of the intersection and the camera’s field of view must be unobstructed. Based on the location and distance of the camera from the desired detection zone, the field of view can be calculated and the required lens can be determined.
Video detection can also be used for special applications where vehicle length and speed measurements are used to warn motorists in advance of roundabouts, off-ramps, and horizontal curves to prevent rollovers or run-off-the-road crashes.

**Figure 5.1 – Video Imaging Detector**

**Figure 5.2 – Active Warning Device along MD 276**

**Microloop Probes (Magneto-inductive Vehicle Sensor)**

A *microloop probe*, or magneto-inductive vehicle sensor, is a small, cylindrical, passive transducer that detects vehicles by sensing disruptions in the Earth’s magnetic field. Non-invasive microloop probes placed underneath the roadway surface in a nonmetallic conduit are SHA’s preferred type of passage detection. Another method of installing microloop probes beneath the roadway surface is by drilling a 1-inch diameter hole (18 to 24 inches deep) in the pavement and inserting the probes (i.e., invasive installation).
The microloop probe set (three probes) has the same general application and placement as a 6-foot by 6-foot inductive loop. However, the probes have better resistance to placing false calls (i.e., when vehicles in adjacent lanes are detected), and the probes have a greater service life over inductive loops due to the reduced exposure to traffic, pavement movement and deterioration, and roadway construction, such as pavement resurfacing.

**Figure 5.3 – Microloop Probe Detector (Plan view)**

**Inductive Loops (Loop detector)**

Inductive loop detectors can be used for both passage and presence detection. The loops are formed by sawcutting the roadway, placing a #14 AWG wire incased in flexible tubing in the sawcut, and then sealing the sawcut. Large area “quadruple” type detectors, typically placed at the stop line, require three sawcuts in the longitudinal direction and the loop wires are wrapped in a “figure 8” pattern.

When used for passage detection, a 6-foot by 6-foot loop is placed in each travel lane in advance of the intersection. Inductive loops used for presence detection, typically 6-foot by 30-foot quadruples, are placed one foot behind the stop line on minor street approaches and in major street left-turn lanes.

**Figure 5.4 – Inductive Loops (Stop line presence detectors)**
Other Technologies

Although less commonly used, several other technologies are available for signal detection, including:

- Infrared
- Light Emission
- Magnetic
- Magnetometer
- Microwave
- Radar
- Sonic

D. Basic Detector Settings

Locking Memory

Passage detectors typically have a setting of locking memory, which “remembers” a vehicle call even when the vehicle is no longer present within the detection zone. With this mode of operation, the controller keeps count of the detector calls during the red interval and retains, or locks, the call until the phase is serviced.

Locking memory should only be used with presence detection where a single vehicle, stopped at an intersection, will not occupy the detection zone but still needs the phase to be called (e.g., where vehicles pull past the stop line because of limited sight distance or large corner radii).

To avoid placing false calls when an approach vehicle enters the intersection during the yellow interval (and consequently serving the phase without any vehicles present), SHA uses a controller setting that enables locking memory only during the red interval (LOCK DETECTORS IN RED ONLY).

Non-Locking Memory

With non-locking memory, detector calls are not retained by the traffic signal controller when the vehicle leaves the detection zone. This means that the controller only recognizes vehicles that remain within the detection zone. Once a vehicle leaves the detection zone the controller “forgets” the vehicle and the call is removed. Presence detectors typically have a non-locking memory setting.

Unnecessary signal changes for certain movements, such as right turns or permissive left turns, can be avoided by utilizing this mode of operation. The non-locking detector mode should also be used in situations where vehicles turn off the major street, cross over unoccupied minor street detection zones, and place false calls into the signal controller.

The memory setting is programmed in the traffic signal controller for each individual detector.
Delay
Delay requires a vehicle to remain within the detection zone for a preset period of time before the call is placed in the traffic signal controller. Detectors that serve exclusive right-turn and left-turn lanes typically utilize the delay feature, which allows the call to be canceled when a vehicle completes the turn within the delay time. The following delay times typically are used by SHA.

- Exclusive/permissive left-turn movements – 4 seconds
- Right-turn movements – 10 to 30 seconds

The delay setting is programmed in the traffic signal controller for each individual detector.

Recall
When calls for conflicting phases are not present, a traffic signal controller typically will rest on the last phase serviced. The recall function in the traffic signal controller can be used to return to a particular phase even without demand. Every phase has the capability of operating with the following types of recall:

- Minimum Recall – Each cycle, a temporary call to service the minimum green interval is placed on the phase. If a vehicle call is received prior to the phase being serviced, the temporary call will be removed. Once the phase is serviced it can be extended based on normal vehicle extension settings.
- Maximum Recall – Each cycle, a constant vehicle call is placed on the phase to service the maximum green interval. Maximum recall typically is used to call a phase when local detection is not present or inoperative.
- Pedestrian Recall – Each cycle, the pedestrian walk and pedestrian clearance intervals are serviced (along with the corresponding vehicular green interval). After the pedestrian intervals are served, normal green timing (i.e., vehicle extension) is in effect; however, the pedestrian phase can be re-serviced only after opposing phases are serviced.
- Soft Recall – Once all calls are serviced each cycle, the signal controller will return to and rest in phases programmed with soft recall, even if demand is not present, until a conflicting call is detected. The major street through phases are typically the “rest phases” when this feature is used.

Detector Cross Switching
Although vehicle detectors are assigned to specific signal phases in the traffic signal controller, there are situations where a single detector can be used to call or extend multiple signal phases. The detector cross switching setting allows a detector to be assigned to different phases by initially assigning the detector to a phase then changing the assignment later in the signal sequence. For example, a vehicle detector for a side street right-turn movement can be used to call the corresponding side street phase then “switched” to extend the major street left-turn phase as part of an overlap right-turn phase.
6. BASIC TIMING PARAMETERS

The calculations and procedures presented in this chapter are used to determine basic signal timing parameters, which depend on traffic composition, roadway geometrics, and the placement of vehicle detection. These parameters are the core settings in the traffic signal controller and are depicted on the first sheet of the SHA Signal Timing Chart. A sample timing chart is presented in Figure 6.1.

![SHA Signal Timing Chart (First Sheet)]

A. Minimum Green Interval – MIN GREEN (Initial Interval)

The minimum green interval, sometimes referred to as the initial interval, is the shortest amount of time that the right-of-way (i.e., green indication) will be assigned to a particular traffic signal phase. It is established to allow vehicles that are stopped on the approach between the detector and the stop line to enter the intersection. Therefore, calculating the time required for this interval depends on the distance between the detection zone and the stop line.

Typically, for major street approaches there is no stop line detection zone – only a “back” (passage) detection zone located several hundred feet in advance of the stop line. In this case, the minimum green interval must be long enough to clear all vehicles that can queue between the stop line and the back detection zone.
Example 6.1: Determine the minimum green intervals for the eastbound and westbound approaches given the following information:

- Eastbound detector-to-stop line distance: 350 feet
- Westbound detector-to-stop line distance: 400 feet

Step 1: Calculate the approximate number of vehicles that can store between the back detection zone and the stop line, \( N \).

**Eastbound:**
\[
N = \frac{350 \text{ ft}}{25 \text{ ft/veh}} = 14 \text{ vehicles}
\]

**Westbound:**
\[
N = \frac{400 \text{ ft}}{25 \text{ ft/veh}} = 16 \text{ vehicles}
\]

Step 2: Calculate the time for the minimum green intervals.

**Eastbound:**
\[
\text{MIN GREEN} = 3.7 + 2.1 \times 14 = 33.1 \text{ sec} \rightarrow 34 \text{ sec}
\]

**Westbound:**
\[
\text{MIN GREEN} = 3.7 + 2.1 \times 16 = 37.3 \text{ sec} \rightarrow 38 \text{ sec}
\]
**Typical Settings**

For major street approaches and high-speed approaches, driver expectancy suggests a minimum green interval of 15 to 30 seconds. For minor street approaches, a minimum green interval of 5 to 8 seconds is typically used, and at locations with high heavy vehicle volumes, or at unusually wide intersections, the minimum green interval should be 8 to 15 seconds. For left-turn phases, a minimum green interval of 3 to 8 seconds is commonly used.

**B. Pedestrian Walk Interval – WALK**

The pedestrian walk interval must be long enough for a small group of pedestrians to enter the intersection and begin the crossing maneuver prior to the start of the pedestrian clearance interval.

SHA’s recommended duration for the pedestrian walk interval is based upon Section 4E.10 of the *Manual on Uniform Traffic Control Devices (MUTCD)*. As set forth in the *MUTCD*, “the walk interval should be at least 7 seconds” so that pedestrians will have adequate opportunity to leave the curb or shoulder before the pedestrian clearance interval begins. If pedestrian volumes and characteristics do not require a 7-second walk interval, walk intervals as short as 4 seconds may be used.

It is important to note that calculations and crosswalk measurements are not required to determine the length of the pedestrian walk interval. Traffic and pedestrian studies should be conducted to evaluate if adjustments to the suggested 7-second walk interval are appropriate – e.g., central business districts with high pedestrian volumes and locations adjacent to schools or elderly care facilities may require longer walk intervals.

**C. Pedestrian Clearance Interval – PED CLEAR (Flashing Don’t Walk Interval)**

The pedestrian clearance interval, or flashing don’t walk interval, must be long enough to allow a pedestrian to walk from curb line to curb line. The United States Access Board (and future revisions/editions of the *MUTCD*) suggests using a walking speed of 3.5 feet per second to calculate the pedestrian clearance interval.

SHA’s recommended practice is that the entire pedestrian clearance interval should be contained within the vehicular green interval because SHA’s policy is to use countdown pedestrian signal displays. In compliance with Section 4E.07 of the *MUTCD*, “Countdown displays shall not be used during the walk interval nor during the yellow change interval of a concurrent vehicular phase.”

### Calculating the pedestrian walk and pedestrian clearance intervals

The following is SHA’s recommended practice for calculating pedestrian intervals.

- **Pedestrian walk interval (WALK) = 7 seconds (no calculation required)**

- **Pedestrian clearance interval (PED CLEAR) =**

  \[
  \text{curb-to-curb} \text{ crosswalk distance} = \frac{3.5 \text{ ft}}{\text{sec}}
  \]
Example 6.2: Determine the pedestrian walk and pedestrian clearance intervals for the east-west pedestrian phase:

- East-west pedestrian phase operates concurrently with the westbound phase

**Step 1:** Identify the time required for the walk interval.

WALK = 7 seconds

**Step 2:** Calculate the time required for the pedestrian clearance interval.

PED CLEAR = \( \frac{63 \text{ feet}}{3.5 \text{ ft/sec}} = 18 \text{ seconds} \)

*Easy technique to check pedestrian intervals in the field*

Time the walk interval to ensure that it is 7 seconds.

At the end of the walk interval/beginning of pedestrian clearance interval, start walking across the crosswalk beginning at the edge of the curb. If the steady don’t walk indication is illuminated before you reach the other side, the pedestrian clearance interval needs to be increased.
**Pedestrian Intervals at Crossings with Wide Medians**

At intersections with wide medians without a median push button, SHA’s preferred duration for the pedestrian clearance interval should be long enough so that pedestrians can cross the entire intersecting roadway on one cycle. Although medians can serve as a “refuge” for pedestrians, requiring two or more cycles for pedestrians to cross could result in a high number of illegal crossings.

At intersections where a push button is provided in the median, the pedestrian clearance interval should be equal to the crossing time (at 3.5 feet per second) required for the longer of the two distances from the curb line to the median, as shown in Figure 6.2.

![Pedestrian Intervals for Medians with Push Buttons](image)

- **Pedestrian walk interval (WALK) = 7 seconds (no calculation required)**
- **Pedestrian clearance interval (PED CLEAR) =**

\[
\text{Higher of: } \frac{X_1}{3.5 \text{ ft/sec}} \text{ or } \frac{X_2}{3.5 \text{ ft/sec}}
\]

**Figure 6.2 – Pedestrian intervals for medians with push buttons**
D. **Vehicle Extension – VEH EXT (Gap or Passage)**

The **vehicle extension interval** is the maximum allowable gap in traffic flow that will retain the green interval for a specific phase. As shown in Figure 6.3, if vehicles are detected at shorter gaps than the vehicle extension interval, then the green interval will be extended for that phase. Gaps in excess of the vehicle extension will cause the green interval to be terminated and the traffic signal controller will proceed to the next traffic phase with demand present.

![Figure 6.3 – Green time extended by vehicle actuations](image)

Safety considerations require that the vehicle extension interval be sufficient enough to permit a vehicle approaching an intersection to pass through the dilemma zone before the yellow change interval begins. The dilemma zone is the area on the approach where the driver, upon seeing the yellow indication, must decide whether to stop or proceed through the intersection. If the vehicle extension interval is too short, abrupt, unsafe stops may result and the phase may be terminated before the movement has been adequately served. If the vehicle extension is too long, excessive delays may result. Generally, stops and delays are minimized when the vehicle extension interval is set to the minimum required value.

The vehicle extension interval should equal the number of seconds required to travel from the back (passage) detection zone to the stop line at the prevailing speed. On approaches with stop line (presence) detectors as well as back detection zones, the vehicle extension interval should equal the time required for a vehicle to travel from the back detection zone to the stop line detection zone at the prevailing speed.
Calculating the vehicle extension interval

The vehicle extension interval is calculated by dividing the distance between the back (passage) detection zone and the stop line (or the distance between the back and front detection zones, if present), by the prevailing speed on the approach.

\[
\text{Vehicle extension interval (VEH EXT)} = \frac{D}{1.47 \times V}
\]

where,

\( D \) = distance between the back detection zone to the stop line (or front detection zone, if present)

\( V \) = velocity of approaching vehicle (MPH) – higher of 85th percentile speed or posted speed limit

1.47 = conversion constant for MPH to ft/sec

Example 6.3: Determine the vehicle extension interval for the eastbound approach at an intersection given the following information:

- Posted speed limit = 50 MPH
- 85th percentile speed = 54 MPH
- Detector-to-stop line distance: 350 feet

Step 1: Identify the approach speed, \( V \), for the calculations.

85th percentile speed = 54 MPH > Posted speed limit = 50 MPH

\( V = 54 \text{ MPH} \)

Step 2: Calculate the time for the vehicle extension interval.

\[
\text{VEH EXT} = \frac{350}{1.47 \times 54} = 4.4 \text{ sec}
\]
**Typical Settings**

Vehicle extension intervals for major street approaches are typically 4 to 7 seconds, while vehicle extension intervals for minor street approaches and left-turn movements are typically 3 to 5 seconds.

Site-specific vehicular and/or roadway characteristics may require longer vehicle extension intervals – e.g., high percentage of heavy vehicles, steep grades, etc. At these locations, SHA’s best practice is to implement the typical timing settings and then review the operations in the field to adjust the timings accordingly.

**Time-of-Day Settings (VEH EXT2)**

Most traffic signal controllers have an alternate vehicle extension interval (VEH EXT2) that can be activated through time-of-day programming. This feature is commonly used along approaches with a high volume of school buses because of their slow acceleration rates. A slightly higher vehicle extension interval is used during specific time periods to prevent early “gap outs” for the phase(s) on that approach.

**Video Detection Adjustments**

The primary difference between timing loop detectors and video detectors is the perspective of a field of view extending from the camera to the detection zone. With an inductive loop, a vehicle is detected when it is over the loop. With video detection, a vehicle is detected when it is within the camera’s field of view. This difference is illustrated in Figure 6.4.

![Figure 6.4 – Detection zone differences between loop and video detectors](image)

As shown in Figure 6.4, the truck is detected and the pulse is on for both the loop and video detectors at approximately the same point, position “A.” When the truck leaves the loop, position “B,” it is no longer detected by the loop and the pulse is off. However, with video detection, the
pulse remains on until the truck reaches the edge of the camera’s field of view, position “C.” This implies that, for a given vehicle length and speed, the video detector’s pulse will be longer than the pulse from a loop detector (i.e., the detection zone is longer with video detection).

The vehicle extension interval should be adjusted to achieve the same functions with video detection as loop detectors. For example, to achieve the same operations as a loop detector with a 3.0-second vehicle extension at 45 MPH, the equivalent interval for a video detector would be 2.2 seconds (assuming a 30-foot mounting height and 300-foot detector-to-camera distance). The following tables, Tables 6.1a – 6.1e, are provided to determine adjusted vehicle extension intervals for video detection.

Table 6.1a – Adjusted vehicle extension intervals for video detection (seconds)  
(20-foot camera height, 100-foot detector-to-camera distance)

<table>
<thead>
<tr>
<th>Approach Speed (MPH)</th>
<th>Traditional loop detector vehicle extension interval (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>25</td>
<td>2.3</td>
</tr>
<tr>
<td>35</td>
<td>2.5</td>
</tr>
<tr>
<td>45</td>
<td>2.6</td>
</tr>
<tr>
<td>55</td>
<td>2.7</td>
</tr>
</tbody>
</table>

1 – The higher of the 85th percentile speed or posted speed limit should be used for the approach speed.

Table 6.1b – Adjusted vehicle extension intervals for video detection (seconds)  
(20-foot camera height, 200-foot detector-to-camera distance)

<table>
<thead>
<tr>
<th>Approach Speed (MPH)</th>
<th>Traditional loop detector vehicle extension interval (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>25</td>
<td>1.6</td>
</tr>
<tr>
<td>35</td>
<td>2.0</td>
</tr>
<tr>
<td>45</td>
<td>2.2</td>
</tr>
<tr>
<td>55</td>
<td>2.4</td>
</tr>
</tbody>
</table>

1 – The higher of the 85th percentile speed or posted speed limit should be used for the approach speed.

Table 6.1c – Adjusted vehicle extension intervals for video detection (seconds)  
(30-foot camera height, 100-foot detector-to-camera distance)

<table>
<thead>
<tr>
<th>Approach Speed (MPH)</th>
<th>Traditional loop detector vehicle extension interval (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>25</td>
<td>2.5</td>
</tr>
<tr>
<td>35</td>
<td>2.7</td>
</tr>
<tr>
<td>45</td>
<td>2.7</td>
</tr>
<tr>
<td>55</td>
<td>2.8</td>
</tr>
</tbody>
</table>

1 – The higher of the 85th percentile speed or posted speed limit should be used for the approach speed.
Table 6.1d – Adjusted vehicle extension intervals for video detection (seconds) (30-foot camera height, 200-foot detector-to-camera distance)

<table>
<thead>
<tr>
<th>Approach Speed (MPH)</th>
<th>Traditional loop detector vehicle extension interval (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>25</td>
<td>2.1</td>
</tr>
<tr>
<td>35</td>
<td>2.4</td>
</tr>
<tr>
<td>45</td>
<td>2.5</td>
</tr>
<tr>
<td>55</td>
<td>2.6</td>
</tr>
</tbody>
</table>

1 – The higher of the 85th percentile speed or posted speed limit should be used for the approach speed.

Table 6.1e – Adjusted vehicle extension intervals for video detection (seconds) (30-foot camera height, 300-foot detector-to-camera distance)

<table>
<thead>
<tr>
<th>Approach Speed (MPH)</th>
<th>Traditional loop detector vehicle extension interval (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>25</td>
<td>1.6</td>
</tr>
<tr>
<td>35</td>
<td>2.0</td>
</tr>
<tr>
<td>45</td>
<td>2.2</td>
</tr>
<tr>
<td>55</td>
<td>2.4</td>
</tr>
</tbody>
</table>

1 – The higher of the 85th percentile speed or posted speed limit should be used for the approach speed.

E. **Yellow Change Interval – YELLOW**

The function of the yellow change interval is to warn motorists of an impending change in the assignment of right-of-way – i.e., notify motorists that a red signal indication will be displayed next (with the exception of exclusive/permissive left-turn signal heads). The yellow change interval should be long enough so motorists can see the indication change from green to yellow and then decide whether to stop or enter the intersection. Specifically, it should be long enough to allow motorists farther away from the intersection to comfortably stop in advance of the intersection or allow motorists closer to the signal to enter the intersection.
Calculating the yellow change interval

The yellow change interval should be calculated using the following formula from the Institute of Transportation Engineers (ITE) *Traffic Engineering Handbook*:

\[
\text{Yellow change interval (YELLOW)} = t + \frac{1.47 \times V}{2(a + Gg)}
\]

where,
- \(t\) = driver perception/reaction time, typically 1.0 seconds
- \(V\) = velocity of approaching vehicle (MPH) – higher of 85\textsuperscript{th} percentile or posted speed
- \(a\) = deceleration rate, typically 10 ft/sec\(^2\)
- \(G\) = acceleration due to gravity, 32 ft/sec\(^2\)
- \(g\) = grade of approach, in percent divided by 100 (downhill is negative)
- 1.47 = conversion constant for MPH to ft/sec

As set forth in Section 4D.10 of the *MUTCD*, the duration of a yellow change interval shall be predetermined and it should have a duration of approximately 3 to 6 seconds; however, SHA does not use yellow change intervals lower than 3.5 seconds. Table 6.2 provides calculated yellow change intervals for various approach speeds and grades.

### Table 6.2 – Calculated yellow change intervals

<table>
<thead>
<tr>
<th>Approach Speed (MPH)(^1)</th>
<th>5% Grade Uphill</th>
<th>Level Terrain</th>
<th>5% Grade Downhill</th>
<th>10% Grade Downhill</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>4.0</td>
</tr>
<tr>
<td>30</td>
<td>3.5</td>
<td>3.5</td>
<td>4.0</td>
<td>4.5</td>
</tr>
<tr>
<td>35</td>
<td>3.5</td>
<td>4.0</td>
<td>4.5</td>
<td>5.0</td>
</tr>
<tr>
<td>40</td>
<td>3.5</td>
<td>4.0</td>
<td>4.5</td>
<td>5.5</td>
</tr>
<tr>
<td>45</td>
<td>4.0</td>
<td>4.5</td>
<td>5.0</td>
<td>6.0</td>
</tr>
<tr>
<td>50</td>
<td>4.5</td>
<td>5.0</td>
<td>5.5</td>
<td>6.0 + 0.5(^\text{AR})</td>
</tr>
<tr>
<td>55</td>
<td>4.5</td>
<td>5.5</td>
<td>6.0</td>
<td>6.0 + 1.0(^\text{AR})</td>
</tr>
<tr>
<td>60</td>
<td>5.0</td>
<td>5.5</td>
<td>6.0 + 0.5(^\text{AR})</td>
<td>6.0 + 1.5(^\text{AR})</td>
</tr>
<tr>
<td>65</td>
<td>5.5</td>
<td>6.0</td>
<td>6.0 + 1.0(^\text{AR})</td>
<td>6.0 + 2.0(^\text{AR})</td>
</tr>
</tbody>
</table>

\(^1\) – The higher of the 85\textsuperscript{th} percentile speed or posted speed limit should be used for the approach speed.

\(^\text{AR}\) – Because the recommended duration of the yellow change interval, based on the ITE formula, exceeds 6.0 seconds, the remainder of the time should be added to the red clearance interval.

### Heavy Vehicle Considerations

If an approach has more than 15 percent heavy vehicles, then a slower deceleration rate, \(a\), should be used to accommodate the reduced stopping ability of heavy vehicles – 8 ft/sec\(^2\) is recommended.
Table 6.3 presents adjusted yellow change intervals for various approach speeds and grades on approaches with heavy vehicle percentages greater than 15 percent.

Table 6.3 – Adjusted yellow change intervals (for approaches with greater than 15% heavy vehicles)

<table>
<thead>
<tr>
<th>Approach Speed (MPH)</th>
<th>5% Grade Uphill</th>
<th>Level Terrain</th>
<th>5% Grade Downhill</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>3.5</td>
<td>3.5</td>
<td>4.0</td>
</tr>
<tr>
<td>30</td>
<td>3.5</td>
<td>4.0</td>
<td>4.5</td>
</tr>
<tr>
<td>35</td>
<td>4.0</td>
<td>4.5</td>
<td>5.0</td>
</tr>
<tr>
<td>40</td>
<td>4.5</td>
<td>5.0</td>
<td>6.0</td>
</tr>
<tr>
<td>45</td>
<td>4.5</td>
<td>5.5</td>
<td>6.0 + 0.5&lt;sup&gt;AR&lt;/sup&gt;</td>
</tr>
<tr>
<td>50</td>
<td>5.0</td>
<td>6.0</td>
<td>6.0 + 1.0&lt;sup&gt;AR&lt;/sup&gt;</td>
</tr>
<tr>
<td>55</td>
<td>5.5</td>
<td>6.0</td>
<td>6.0 + 1.5&lt;sup&gt;AR&lt;/sup&gt;</td>
</tr>
<tr>
<td>60</td>
<td>6.0</td>
<td>6.0 + 0.5&lt;sup&gt;AR&lt;/sup&gt;</td>
<td>6.0 + 2.0&lt;sup&gt;AR&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

1 – The higher of the 85<sup>th</sup> percentile speed or posted speed limit should be used for the approach speed.

<sup>AR</sup> – Because the recommended duration of the yellow change interval, based on the ITE formula, exceeds 6.0 seconds, the remainder of the time should be added to the red clearance interval.

Other Considerations

Transportation professionals should consider several other aspects of intersection operations to determine the appropriate duration of the yellow change interval.

Driver expectation

Because motorists have some expectation of what the length of the yellow change interval should be based on past driving experiences, the yellow change interval should be constant along corridors or signalized arterials. Where feasible, the longest required yellow change interval along the corridor should be used for the major street approaches.

Implementation

A uniform yellow change interval should be used for both approaches of a roadway to an intersection. For example, when one approach has a positive approach grade, and the other approach has a negative approach grade, the more conservative (i.e., longer) yellow change interval should be used for both approaches.

Left-turn phasing

When a separate left-turn phase is used along an approach, the duration of the yellow change interval for the left-turn phase should be the same as the interval for the through phase in the same direction. Although this is a more conservative approach, the yellow change interval for the left-turn phase can be shortened, if necessary.

Additionally, if a lag left-turn phase operates concurrently with a through phase in the same direction, then the greater of the two values for the yellow change interval (calculated separately) should be used for both movements.
**Example 6.4:** Determine the yellow change interval for the eastbound approach to an intersection given the following information:

- Posted speed limit = 50 MPH
- 85th percentile speed = 54 MPH
- 3% downhill grade
- Intersection width (across north/south legs) = 80 feet

**Step 1:** Identify the approach speed, $V$, for the calculations.

$$85\text{th percentile speed} = 54 \text{ MPH} > \text{Posted speed limit} = 50 \text{ MPH}$$

$$V = 54 \text{ MPH}$$

**Step 2:** Calculate the time for the yellow change interval.

$$\text{YELLOW} = 1.0 + \frac{1.47 \times 54}{2 \times [10 + (32.2 \times -0.03)]} = 5.4 \text{ sec} \rightarrow 5.5 \text{ sec}$$

**F. Red Clearance Interval – RED CLEAR**

The red clearance interval provides vehicles that could not stop during the yellow change interval (i.e., vehicles that entered the intersection at the end of the yellow change interval) with additional time to clear the intersection before conflicting traffic enters the intersection.

**Calculating the red clearance interval**

The red clearance interval should be calculated using the following formula from the Institute of Transportation Engineers (ITE) *Traffic Engineering Handbook*:

$$\text{Red clearance interval (RED CLEAR)} = \frac{W + L}{1.47 \times V}$$

where,

- $W$ = curb-to-curb width of the intersection (ft)
- $L$ = length of vehicle, typically 20 feet
- $V$ = **posted speed limit** on the approaching roadway (MPH)
- $1.47 = $ conversion constant for MPH to ft/sec

The red clearance interval can be increased for special situations where reaction times are observed to exceed the typical value of one second, or where turning vehicles typically remain in the intersection at the termination of the green and yellow change intervals and need additional time to clear the intersection (e.g., permissive left turns).
Section 4D.10 of the MUTCD states that the “red clearance interval should have a duration not exceeding 6 seconds,” and the sum of the yellow change and red clearance intervals generally should not exceed 9 seconds. However, red clearance intervals at single-point urban diamond interchanges may need to be extended due to the large intersection area and long vehicle paths in order to safely clear vehicles in the intersection. Red clearance intervals at these locations typically range from 6 to 12 seconds.

**Example 6.5:** Determine the red clearance interval for the eastbound approach to an intersection given the following information:

- Posted speed limit = 50 MPH
- 85th percentile speed = 54 MPH
- 3% downhill grade
- Intersection width (across north/south legs) = 80 feet

**Step 1:** Identify the approach speed, $V$, for the calculations.

$$ V = 50 \text{ MPH} $$

**Step 2:** Calculate the time for the red clearance interval.

$$ \text{RED CLEAR} = \frac{80 + 20}{1.47 \times 50} = 1.4 \text{ sec} \rightarrow 1.5 \text{ sec} $$

**G. Maximum Green Interval – MAX I, MAX II, MAX III**

The *maximum green interval* is the longest time that a particular phase can receive its assignment of right-of-way when demand is present on conflicting phases. This value can be varied by time-of-day programming or it can be calculated “real time” using expanded controller capabilities.

This section presents three different methods for manually determining the duration of the maximum green intervals for each signal phase:

1. Proportional V/C ratio method
2. Greenshields’ green time method
3. Poisson distribution method

All of the approaches are based upon the critical lane volume (CLV) analysis, presented in Chapter 2, and use a background cycle length to approximate the intersection’s cycle-by-cycle operations.
**Background Cycle Length**

The *cycle length* is the amount of time needed for the traffic signal controller to complete one sequence of all of the traffic signal phases, including the yellow change and red clearance intervals. Although the cycle length is not a direct input into isolated, actuated traffic signal controllers, the determination of a reasonable cycle length (i.e., *background cycle length*) serves as a preliminary step to calculate the maximum green intervals for each phase.

Table 6.4 presents SHA’s recommended maximum cycle lengths for various levels of service, or total intersection critical lane volume, depending upon the number of conflicting movements.

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>2 Conflicting Movements</th>
<th>3 Conflicting Movements</th>
<th>4 Conflicting Movements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2 phases</td>
<td>3 – 5 phases</td>
<td>6 – 8 phases</td>
</tr>
<tr>
<td>A CLV &lt; 1,000 vph</td>
<td>90</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>B 1,000 ≤ CLV ≤ 1,150 vph</td>
<td>90</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>C 1,151 ≤ CLV ≤ 1,300 vph</td>
<td>100</td>
<td>120</td>
<td>135</td>
</tr>
<tr>
<td>D 1,301 ≤ CLV ≤ 1,450 vph</td>
<td>120</td>
<td>135</td>
<td>150</td>
</tr>
<tr>
<td>E 1,451 ≤ CLV ≤ 1,600 vph</td>
<td>135</td>
<td>150</td>
<td>165</td>
</tr>
<tr>
<td>F CLV &gt; 1,600 vph</td>
<td>150</td>
<td>165</td>
<td>180</td>
</tr>
</tbody>
</table>

**Webster’s formula**

Determining the “optimal” cycle length requires balancing two competing characteristics – capacity and delays/queues. With shorter cycle lengths, a larger percentage of the cycle is consumed by the yellow change and red clearance intervals. Additionally, a higher percentage of the cycle length is occupied by start-up (lost) time for each movement with shorter cycles. With longer cycle lengths, there is more effective green time per hour, but longer cycles can result in increased delays and longer queues.

An alternative method for determining the background cycle length is Webster’s formula, which minimizes intersection delay based on total lost time (i.e., change and clearance intervals) and critical lane volume.
If a new signal is installed along an existing signal system, then the background cycle length for calculating the maximum green intervals should equal the longest cycle length currently in operation within the signal system.

**Method 1: Proportional Volume-to-Capacity (V/C) Ratio Method**

This approach apportions the total green time available per cycle based upon the ratio, or percentage, of each phase’s critical lane volume and the total intersection critical lane volume.

**Calculating the maximum green interval: Proportional V/C Ratio Method**

1. Using turning movement counts, determine the highest 60-minute volume for each phase. **NOTE: The volumes for each movement do not need to be from the same 60-minute period.**
2. Calculate the critical lane volume for each phase.
3. Calculate the total intersection critical lane volume and identify the critical phases.
4. From Table 6.4, or using Webster’s formula, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.
5. Sum the yellow change intervals for the critical phases, \( Y_{\text{TOTAL}} \).
6. Sum the red clearance intervals for the critical phases, \( R_{\text{TOTAL}} \).
7. Calculate the total available green time (per cycle), \( G_{\text{TOTAL}} \), using the background cycle length (Step 4), \( Y_{\text{TOTAL}} \) (Step 5), and \( R_{\text{TOTAL}} \) (Step 6).

\[
G_{\text{TOTAL}} = \text{Background cycle length} - Y_{\text{TOTAL}} - R_{\text{TOTAL}}
\]

8. For each phase, apportion the total available green time (per cycle), \( G_{\text{TOTAL}} \) (Step 7), based upon the ratio of critical lane volume for each phase (Step 2) and the total intersection critical lane volume (Step 3).

9. Maximum green interval (MAX I) = \[
\frac{G_{\text{TOTAL}} \times \text{Phase CLV}}{\text{Total intersection CLV}}
\]
Example 6.6: Determine the maximum green interval for each phase using the proportional V/C ratio method given the following information:

- Phasing sequence – exclusive left-turn phasing (major street) and split side-street phasing

- Lane use factor (LUF) for double left-turn lanes: 0.60 (see Chapter 11)

- Change and clearance intervals:
  - Phase 1 & Phase 5 yellow change intervals: 3 sec
  - Phase 2 & Phase 6 yellow change intervals: 5 sec
  - Phase 3 & Phase 4 yellow change intervals: 4 sec
  - Red clearance intervals (all phases): 2 sec

Step 1: Using turning movement counts, determine the highest 60-minute volume for each phase.

See given turning movement volumes.

Step 2: Calculate the critical lane volume for each phase.

- Phase 1 (NB left): CLV = 450 × 0.60 = 270 vph
- Phase 2 (SB through): CLV = 515 vph
- Phase 3 (WB left & through): CLV = max(150, 300) = 300 vph
- Phase 4 (EB left & through): CLV = max(215, 375) = 375 vph
- Phase 5 (SB left): CLV = 350 × 0.60 = 210 vph
- Phase 6 (NB through): CLV = 500 vph
Step 3: Calculate the total intersection critical lane volume.

(NB left + SB through): CLV = 270 + 515 = 785 vph *critical
(SB left + NB through): CLV = 210 + 500 = 710 vph
(WB left & through): CLV = 300 vph *critical
(EB left & through): CLV = 375 vph *critical

Total intersection critical lane volume = 785 + 300 + 375 = 1,460 vph (LOS E)

Step 4: From Table 6.4, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.

From Table 6.4 (1,460 vph, 4 conflicting movements) → 165-sec background cycle length

Step 5: Sum the yellow change intervals for the critical phases, Y_{TOTAL}.

From Step 3, the critical phases are Phases 1, 2, 3, & 4.

Y_{TOTAL} = 3 + 5 + 4 + 4 = 16 sec

Step 6: Sum the red clearance intervals for the critical phases, R_{TOTAL}.

From Step 3, the critical phases are Phases 1, 2, 3, & 4.

R_{TOTAL} = 2 + 2 + 2 + 2 = 8 sec

Step 7: Calculate the total available green time (per cycle), G_{TOTAL}, using the background cycle length (Step 4), Y_{TOTAL} (Step 5), and R_{TOTAL} (Step 6).

G_{TOTAL} = 165 – 16 – 8 = 141 sec

Step 8: For each phase, apportion the total available green time (per cycle), G_{TOTAL} (Step 7), based upon the ratio of critical lane volume for each phase (Step 2) and the total intersection critical lane volume (Step 3).

Phase 1 (NB left): 141 sec × (270 vph ÷ 1,460 vph) = 26.1 sec
Phase 2 (SB through): 141 sec × (515 vph ÷ 1,460 vph) = 49.7 sec
Phase 3 (WB left & through): 141 sec × (300 vph ÷ 1,460 vph) = 29.0 sec
Phase 4 (EB left & through): 141 sec × (375 vph ÷ 1,460 vph) = 36.2 sec
Phase 5 (SB left): CLV = 141 sec × (210 vph ÷ 1,460 vph) = 20.3 sec
Phase 6 (NB through): CLV = 141 sec × (500 vph ÷ 1,460 vph) = 48.3 sec
Method 2: Greenshields’ Green Time (Vehicle Headway) Method

This approach determines the amount of green time required to serve the number of “critical” vehicles arriving each cycle for each phase. The green time requirements, presented in Table 6.5, are based on Bruce Greenshields’ research on vehicle headways and saturation flow rates at intersections.

Table 6.5 – Green time requirements (Greenshields’ vehicle headway model)

<table>
<thead>
<tr>
<th>Avg. Vehicles per Cycle per Lane</th>
<th>Green Time per Vehicle</th>
<th>Cumulative Green Time</th>
<th>Avg. Vehicles per Cycle per Lane</th>
<th>Green Time per Vehicle</th>
<th>Cumulative Green Time</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>3.8</td>
<td>3.8</td>
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<td>2.1</td>
<td>47.8</td>
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<tr>
<td>2</td>
<td>3.1</td>
<td>6.9</td>
<td>22</td>
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<td>49.9</td>
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<tr>
<td>4</td>
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<td>56.2</td>
</tr>
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<td>16.3</td>
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<td>58.3</td>
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<td>73.0</td>
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<td>34</td>
<td>2.1</td>
<td>75.1</td>
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<td>35.2</td>
<td>35</td>
<td>2.1</td>
<td>77.2</td>
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<tr>
<td>16</td>
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<td>37.3</td>
<td>36</td>
<td>2.1</td>
<td>79.3</td>
</tr>
<tr>
<td>17</td>
<td>2.1</td>
<td>39.4</td>
<td>37</td>
<td>2.1</td>
<td>81.4</td>
</tr>
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<td>41.5</td>
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<td>83.5</td>
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<td>19</td>
<td>2.1</td>
<td>43.6</td>
<td>39</td>
<td>2.1</td>
<td>85.6</td>
</tr>
<tr>
<td>20</td>
<td>2.1</td>
<td>45.7</td>
<td>40</td>
<td>2.1</td>
<td>87.7</td>
</tr>
</tbody>
</table>
Calculating the maximum green interval: Greenshields’ Method

1. Using turning movement counts, determine the highest 60-minute volume for each phase.  
   **NOTE:** The volumes for each movement do not need to be from the same 60-minute period.
2. Calculate the critical lane volume for each phase.
3. Calculate the total intersection critical lane volume.
4. From Table 6.4, or using Webster’s formula, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.
5. Calculate the average number of vehicles per cycle (per lane) for each phase using the critical lane volume (Step 2) and the background cycle length (Step 4).

   \[
   \text{Average vehicles per cycle (per lane)} = \frac{\text{Critical lane volume} \times \text{Background cycle length}}{3,600 \text{ sec/hr}}
   \]

6. Using Table 6.5, determine the cumulative green time required to serve the average vehicles per cycle for each phase (Step 5).
Example 6.7: Determine the maximum green interval for each phase using Greenshields' method given the following information:

- Phasing sequence – exclusive left-turn phasing on all approaches

- Lane use factor (LUF) for double left-turn lanes: 0.60 (see Chapter 11)

Step 1: Using turning movement counts, determine the highest 60-minute volume for each phase.

- See given turning movement volumes.

Step 2: Calculate the critical lane volume for each phase.

- Phase 1 (NB left): CLV = 450 \times 0.60 = 270 vph
- Phase 2 (SB through): CLV = 515 vph
- Phase 3 (EB left): CLV = 215 vph
- Phase 4 (WB through): CLV = 300 vph
- Phase 5 (SB left): CLV = 350 \times 0.60 = 210 vph
- Phase 6 (NB through): CLV = 500 vph
- Phase 7 (WB left): CLV = 100 vph
- Phase 8 (EB through): CLV = 375 vph

Step 3: Calculate the total intersection critical lane volume.

- (NB left + SB through): CLV = 270 + 515 = 785 vph  \textit{critical}
- (SB left + NB through): CLV = 210 + 500 = 710 vph
- (EB left + WB through): CLV = 215 + 300 = 515 vph  \textit{critical}
- (WB left + EB through): CLV = 100 + 375 = 475 vph
- Total intersection critical lane volume = 785 + 515 = 1,300 vph (LOS C)
Step 4: From Table 6.4, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.

From Table 6.4 (1,300 vph, 4 conflicting movements) → 135-sec background cycle length

Step 5: Calculate the average vehicles per cycle (per lane) for each phase using the critical lane volume (Step 2) and the background cycle length (Step 4).

Phase 1 (NB left): CLV = 270 vph × (135 sec ÷ 3600) = 10 veh/cycle
Phase 2 (SB through): CLV = 515 vph × (135 sec ÷ 3600) = 19 veh/cycle
Phase 3 (EB left): CLV = 215 vph × (135 sec ÷ 3600) = 8 veh/cycle
Phase 4 (WB through): CLV = 300 vph × (135 sec ÷ 3600) = 11 veh/cycle
Phase 5 (SB left): CLV = 210 vph × (135 sec ÷ 3600) = 8 veh/cycle
Phase 6 (NB through): CLV = 500 vph × (135 sec ÷ 3600) = 19 veh/cycle
Phase 7 (WB left): CLV = 100 vph × (135 sec ÷ 3600) = 4 veh/cycle
Phase 8 (EB through): CLV = 375 vph × (135 sec ÷ 3600) = 14 veh/cycle

Step 6: Using Table 6.5, determine the cumulative green time required to serve the average vehicles per cycle for each phase (Step 5).

Phase 1 (NB left): 10 veh/cycle → 24.7 sec
Phase 2 (SB through): 19 veh/cycle → 43.6 sec
Phase 3 (EB left): 8 veh/cycle → 20.5 sec
Phase 4 (WB through): 11 veh/cycle → 26.8 sec
Phase 5 (SB left): 8 veh/cycle → 20.5 sec
Phase 6 (NB through): 19 veh/cycle → 43.6 sec
Phase 7 (WB left): 4 veh/cycle → 12.0 sec
Phase 8 (EB through): 14 veh/cycle → 33.1 sec
Method 3: Poisson Distribution Method

The Poisson distribution method, similar to Greenshields' method, is based upon determining the amount of green time required to serve the number of “critical” vehicles arriving each cycle. However, as opposed to using the average number of vehicles to determine the green time requirements, this approach uses the Poisson distribution (i.e., random vehicle arrivals) to calculate the maximum (95th percentile) number of vehicles that will arrive each cycle.

Table 6.6 presents the maximum vehicles per cycle from the Poisson distribution and the corresponding green time requirements using Greenshields' vehicle headway model.

<table>
<thead>
<tr>
<th>Avg. Vehicles per Cycle per Lane</th>
<th>Max No. of Vehicles per Cycle per Lane</th>
<th>Cumulative Green Time (Greenshields')</th>
<th>Avg. Vehicles per Cycle per Lane</th>
<th>Max No. of Vehicles per Cycle per Lane</th>
<th>Cumulative Green Time (Greenshields')</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 – 0.3</td>
<td>1</td>
<td>3.8</td>
<td>14.1 – 14.9</td>
<td>21</td>
<td>47.8</td>
</tr>
<tr>
<td>0.4 – 0.8</td>
<td>2</td>
<td>6.9</td>
<td>15.0 – 15.7</td>
<td>22</td>
<td>49.9</td>
</tr>
<tr>
<td>0.9 – 1.3</td>
<td>3</td>
<td>9.6</td>
<td>15.8 – 16.5</td>
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<td>52.0</td>
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<td>1.4 – 1.9</td>
<td>4</td>
<td>12.0</td>
<td>16.6 – 17.3</td>
<td>24</td>
<td>54.1</td>
</tr>
<tr>
<td>2.0 – 2.6</td>
<td>5</td>
<td>14.2</td>
<td>17.4 – 18.2</td>
<td>25</td>
<td>56.2</td>
</tr>
<tr>
<td>2.7 – 3.2</td>
<td>6</td>
<td>16.3</td>
<td>18.3 – 19.0</td>
<td>26</td>
<td>58.3</td>
</tr>
<tr>
<td>3.3 – 3.9</td>
<td>7</td>
<td>18.4</td>
<td>19.1 – 19.8</td>
<td>27</td>
<td>60.4</td>
</tr>
<tr>
<td>4.0 – 4.7</td>
<td>8</td>
<td>20.5</td>
<td>20.0 – 20.7</td>
<td>28</td>
<td>62.5</td>
</tr>
<tr>
<td>4.8 – 5.4</td>
<td>9</td>
<td>22.6</td>
<td>20.8 – 21.5</td>
<td>29</td>
<td>64.6</td>
</tr>
<tr>
<td>5.5 – 6.1</td>
<td>10</td>
<td>24.7</td>
<td>21.6 – 22.4</td>
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<td>66.7</td>
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<td>6.2 – 6.9</td>
<td>11</td>
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<td>22.5 – 23.2</td>
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<td>68.8</td>
</tr>
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<td>7.0 – 7.7</td>
<td>12</td>
<td>28.9</td>
<td>23.3 – 24.1</td>
<td>32</td>
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<td>7.8 – 8.4</td>
<td>13</td>
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</tr>
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<td>25.1 – 25.8</td>
<td>34</td>
<td>75.1</td>
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<td>35.2</td>
<td>25.9 – 26.7</td>
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</tr>
<tr>
<td>10.1 – 10.8</td>
<td>16</td>
<td>37.3</td>
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<td>79.3</td>
</tr>
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<td>10.9 – 11.6</td>
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<td>11.7 – 12.4</td>
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<td>83.5</td>
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<td>12.5 – 13.2</td>
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<td>85.6</td>
</tr>
<tr>
<td>13.3 – 14.0</td>
<td>20</td>
<td>45.7</td>
<td>30.2 – 31.0</td>
<td>40</td>
<td>87.7</td>
</tr>
</tbody>
</table>
Calculating the maximum green interval: Poisson Distribution Method

1. Using turning movement counts, determine the highest 60-minute volume for each phase. 
   \textit{NOTE: The volumes for each movement do not need to be from the same 60-minute period.}
2. Calculate the critical lane volume for each phase.
3. Calculate the total intersection critical lane volume.
4. From Table 6.4, or using Webster’s formula, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.
5. Calculate the average number of vehicles per cycle (per lane) for each phase using the critical lane volume (Step 2) and the background cycle length (Step 4).

\[
\text{Average vehicles per cycle (per lane)} = \frac{\text{Critical lane volume} \times \text{Background cycle length}}{3,600 \text{ sec/hr}}
\]

6. Using Table 6.6, determine the maximum (95\textsuperscript{th} percentile) number of vehicles per cycle for each phase and cumulative green time required to serve those vehicles.
**Example 6.8:** Determine the maximum green interval for each phase using the Poisson distribution method given the following information:

- **Phasing sequence** – exclusive left-turn phasing on all approaches

- **Lane use factor (LUF) for double left-turn lanes:** 0.60 (see Chapter 11)

**Step 1:** Using turning movement counts, determine the highest 60-minute volume for each phase.

*See given turning movement volumes.*

**Step 2:** Calculate the critical lane volume for each phase.

- Phase 1 (NB left): \( CLV = 390 \times 0.60 = 234 \text{ vph} \)
- Phase 2 (SB through): \( CLV = 420 \text{ vph} \)
- Phase 3 (EB left): \( CLV = 115 \text{ vph} \)
- Phase 4 (WB through): \( CLV = 150 \text{ vph} \)
- Phase 5 (SB left): \( CLV = 280 \times 0.60 = 168 \text{ vph} \)
- Phase 6 (NB through): \( CLV = 430 \text{ vph} \)
- Phase 7 (WB left): \( CLV = 80 \text{ vph} \)
- Phase 8 (EB through): \( CLV = 175 \text{ vph} \)

**Step 3:** Calculate the total intersection critical lane volume.

- (NB left + SB through): \( CLV = 234 + 420 = 654 \text{ vph} \) *critical*
- (SB left + NB through): \( CLV = 168 + 430 = 598 \text{ vph} \)
- (EB left + WB through): \( CLV = 115 + 150 = 265 \text{ vph} \) *critical*
- (WB left + EB through): \( CLV = 80 + 175 = 255 \text{ vph} \)

Total intersection critical lane volume = 654 + 265 = 919 vph (LOS A)
Step 4: From Table 6.4 determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.

From Table 6.4 (919 vph, 4 conflicting movements) → 120-sec background cycle length

Step 5: Calculate the average vehicles per cycle (per lane) for each phase using the critical lane volume (Step 2) and the background cycle length (Step 4).

<table>
<thead>
<tr>
<th>Phase</th>
<th>CLV (vph)</th>
<th>BCL (sec)</th>
<th>Average Vehicles per Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1 (NB left)</td>
<td>234 vph</td>
<td>120 sec</td>
<td>7.8 veh/cycle</td>
</tr>
<tr>
<td>Phase 2 (SB through)</td>
<td>420 vph</td>
<td>120 sec</td>
<td>14.0 veh/cycle</td>
</tr>
<tr>
<td>Phase 3 (EB left)</td>
<td>115 vph</td>
<td>120 sec</td>
<td>3.8 veh/cycle</td>
</tr>
<tr>
<td>Phase 4 (WB through)</td>
<td>150 vph</td>
<td>120 sec</td>
<td>5.0 veh/cycle</td>
</tr>
<tr>
<td>Phase 5 (SB left)</td>
<td>168 vph</td>
<td>120 sec</td>
<td>5.6 veh/cycle</td>
</tr>
<tr>
<td>Phase 6 (NB through)</td>
<td>430 vph</td>
<td>120 sec</td>
<td>14.3 veh/cycle</td>
</tr>
<tr>
<td>Phase 7 (WB left)</td>
<td>80 vph</td>
<td>120 sec</td>
<td>2.7 veh/cycle</td>
</tr>
<tr>
<td>Phase 8 (EB through)</td>
<td>175 vph</td>
<td>120 sec</td>
<td>5.8 veh/cycle</td>
</tr>
</tbody>
</table>

Step 6: Using Table 6.6, determine the maximum (95th percentile) number of vehicles per cycle for each phase and cumulative green time required to serve those vehicles.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Average Vehicles per Cycle</th>
<th>Maximum Vehicles per Cycle</th>
<th>Green Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1 (NB left)</td>
<td>7.8 veh/cycle</td>
<td>13 max veh/cycle</td>
<td>31.0 sec</td>
</tr>
<tr>
<td>Phase 2 (SB through)</td>
<td>14.0 veh/cycle</td>
<td>20 max veh/cycle</td>
<td>45.7 sec</td>
</tr>
<tr>
<td>Phase 3 (EB left)</td>
<td>3.8 veh/cycle</td>
<td>7 max veh/cycle</td>
<td>18.4 sec</td>
</tr>
<tr>
<td>Phase 4 (WB through)</td>
<td>5.0 veh/cycle</td>
<td>9 max veh/cycle</td>
<td>22.6 sec</td>
</tr>
<tr>
<td>Phase 5 (SB left)</td>
<td>5.6 veh/cycle</td>
<td>10 max veh/cycle</td>
<td>24.7 sec</td>
</tr>
<tr>
<td>Phase 6 (NB through)</td>
<td>14.3 veh/cycle</td>
<td>21 max veh/cycle</td>
<td>47.8 sec</td>
</tr>
<tr>
<td>Phase 7 (WB left)</td>
<td>2.7 veh/cycle</td>
<td>6 max veh/cycle</td>
<td>16.3 sec</td>
</tr>
<tr>
<td>Phase 8 (EB through)</td>
<td>5.8 veh/cycle</td>
<td>10 max veh/cycle</td>
<td>24.7 sec</td>
</tr>
</tbody>
</table>

Typical Settings

For major street through phases, a maximum green interval of 30 to 60 seconds is commonly used. For minor street phases and major street left-turn phases, a maximum green interval of 20 to 30 seconds is typical.
CHAPTER 7

Volume-Density Timing Parameters

Interim Edition
7. VOLUME-DENSITY TIMING PARAMETERS

Volume-density control uses advanced detector and controller functions to help improve the efficiency of actuated control along approaches with long distances between the stop line and passage detectors. SHA recommends considering the use of volume-density control at intersections with approach speeds greater than 45 MPH and minimum green intervals in excess of 25 seconds based on the methods and calculations presented in Chapter 6.

Volume-density timing parameters are depicted on the first sheet of the SHA Signal Timing Chart, as shown in Figure 7.1.

Figure 7.1 – Volume-density timing parameters on SHA Signal Timing Chart

The two primary operational functions of volume-density control are the variable initial and reduced gap (vehicle extension) intervals.

A. Variable Initial (Added Initial or Variable Minimum Green)

With standard actuated control, the duration of the minimum green (initial) interval is typically equal to the amount of time required to service the vehicles queued between the detector and the stop line. Along high-speed approaches (i.e., approaches above 45 MPH), the distance between the passage detector(s) and the stop line can potentially store a large number of vehicles, which can lead to a long minimum green interval and inefficiencies when queues do not extend to the passage detectors.
detector(s).

Because passage detectors are capable of “remembering” the number of queued vehicles along an approach, the initial green interval can be adjusted cycle-by-cycle with volume-density control to correspond to the specific number of queued vehicles along an approach. This is accomplished by using a lower minimum green interval and then increasing the interval based on the number of vehicle actuations received during the red interval. Specifically, the variable initial operation uses the following timing parameters:

- Minimum green interval (MIN GREEN)
- Actuations before addition (ACT B4)
- Seconds per actuation (SEC/ACT)
- Maximum initial (MAX INI)

The variable initial concept is illustrated in Figure 7.2.

![Figure 7.2 – Variable initial timing](image)

**Minimum Green Interval – MIN GREEN**

With volume-density control, the minimum green interval should be set to a lower value such as 15 to 25 seconds or the interval can be calculated based on the time required to serve the typical queue during off-peak traffic conditions.
**Calculating the minimum green interval for volume-density control**

The following formula should be used to calculate the minimum green interval with volume-density control:

\[
\text{Minimum green interval (MIN GREEN)} = 3.7 + 2.1 \times Q
\]

where,

- \( Q \) = number of vehicles per lane expected during off-peak traffic conditions
- 3.7 sec = sum of the lost time (i.e., start-up time) for the first five vehicles in the queue to depart the intersection
- 2.1 sec = average headway for vehicles traveling through an intersection

**Example 7.1:** Determine the minimum green intervals for the eastbound and westbound approaches given the following information:

- Typical off-peak queue:
  - Eastbound approach: 6 vehicles
  - Westbound approach: 8 vehicles

**Step 1:** Calculate the time for the minimum green intervals.

- Eastbound: \( \text{MIN GREEN} = 3.7 + 2.1 \times 6 = 16.3 \text{ sec} \rightarrow 17 \text{ sec} \)
- Westbound: \( \text{MIN GREEN} = 3.7 + 2.1 \times 8 = 20.5 \text{ sec} \rightarrow 21 \text{ sec} \)

**Actuations before Addition – ACT B4**

This timing parameter is the number of actuations received during the red interval before time is added to the minimum green interval. This value should be equal to the total number of vehicles (i.e., all travel lanes) that can be serviced during the “lowered” minimum green interval.

**Calculating the actuations before addition interval**

The following formula should be used to calculate the actuations before addition interval:

\[
\text{Actuations before addition interval (ACT B4)} = \frac{\text{MIN GREEN} - 3.7}{2.1} \times L
\]

where,

- \( L \) = number of lanes
- 3.7 sec = sum of the lost time (i.e., start-up time) for the first five vehicles in the queue to depart the intersection
- 2.1 sec = average headway for vehicles traveling through an intersection

Table 7.1 provides calculated actuations before addition intervals for various minimum green intervals and lane configurations.
Table 7.1 – Calculated actuations before addition intervals

<table>
<thead>
<tr>
<th>Minimum Green (sec.)</th>
<th>No. of Lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>15</td>
<td>5</td>
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<tr>
<td>20</td>
<td>8</td>
</tr>
<tr>
<td>25</td>
<td>10</td>
</tr>
<tr>
<td>30</td>
<td>13</td>
</tr>
</tbody>
</table>

Seconds per Actuation (or Added Initial per Actuation) – SEC/ACT

This timing parameter is the amount of time added to the minimum initial for each vehicle actuation during the red interval. For a single-lane approach, the seconds per actuation is typically equal to 2 seconds. For two-lane and three-lane approaches, the interval should range between 1.0 to 1.5 seconds.

Maximum Initial – MAX INI

The maximum initial interval is the upper limit of the variable initial time. If additional vehicle actuations are received during the red interval and the initial period has reached the maximum, then additional time will not be added. Therefore, the maximum initial interval should be long enough to clear all vehicles that can queue between the stop line and the passage detection zone.

Calculating the maximum initial interval

The following formula should be used to calculate the maximum initial interval:

\[
\text{Maximum initial interval (MAX INI)} = 3.7 + 2.1 \times N
\]

where,

- \(N\) = maximum number of vehicles per lane that can store between the stop line and the back detection zone (assume 25 feet per vehicle)
- 3.7 sec = sum of the lost time (i.e., start-up time) for the first five vehicles in the queue to depart the intersection
- 2.1 sec = average headway for vehicles traveling through an intersection
Example 7.2: Determine the maximum initial intervals for the eastbound and westbound approaches given the following information:

- Eastbound detector-to-stop line distance: 350 feet
- Westbound detector-to-stop line distance: 400 feet

Step 1: Calculate the approximate number of vehicles that can store between the back detection zone and the stop line, \( N \).

\[
\text{Eastbound: } N = \frac{350 \text{ ft}}{25 \text{ ft/veh}} = 14 \text{ vehicles}
\]

\[
\text{Westbound: } N = \frac{400 \text{ ft}}{25 \text{ ft/veh}} = 16 \text{ vehicles}
\]

Step 2: Calculate the time for the maximum initial intervals.

- Eastbound: \( \text{MAX INI} = 3.7 + 2.1 \times 14 = 33.1 \text{ sec} \rightarrow 34 \text{ sec} \)
- Westbound: \( \text{MAX INI} = 3.7 + 2.1 \times 16 = 37.3 \text{ sec} \rightarrow 38 \text{ sec} \)

B. Gap Reduction

Gap reduction reduces the allowable headway (or gap) between vehicles from the preset vehicle extension interval to a lower value over a specified amount of time. Although gap reduction is not commonly used, it can be a valuable tool to improve efficiency when a longer vehicle extension interval is required at an intersection – e.g., approaches with a high proportion of heavy vehicles or buses. Although the longer vehicle extension interval is used to accommodate the slow “start up” time for heavy vehicles, the extended interval often causes phases to be retained even when traffic demand decreases instead of “gapping out.”

Gap reduction consists of the following parameters:

- Vehicle extension (VEH EXT)
- Time before reduction (TIME B4)
- Cars waiting before reduction (CARS WT)
- Time to reduce (TTREDUC)
- Minimum gap (MIN GAP)

The gap reduction concept is illustrated in Figure 7.3.
Time before Reduction – TIME B4
This parameter is the length of time required during the green interval before reducing the gap from the vehicle extension to the minimum gap. The time before reduction begins after the start of the green interval when the first call for a conflicting phase is placed on the controller. Values equal to the minimum green interval or one-third to one-half of the maximum green time are commonly used for this parameter.

Cars Waiting – CARS WT
Cars waiting is the number of actuations required for a conflicting phase before reducing the gap from the vehicle extension to the minimum gap. If both TIME B4 and CARS WT are used, the start of the gap reduction will be initiated when either of the intervals reaches its maximum value.

Time to Reduce – TTREDUC
Time to reduce is the length of time over which the vehicle extension is reduced to the minimum gap. A value equal to one-third to one-half of the maximum green time is commonly used for this parameter.

Minimum Gap – MIN GAP
The minimum gap is the shortest allowable time between vehicle actuations that will retain the green interval for a phase. The interval should be long enough to allow a vehicle traveling at the prevailing approach speed to travel from the passage detector through the dilemma zone. Generally, the minimum gap should not be less than two seconds.
CHAPTER 8
Time-of-Day Timing Parameters

Interim Edition
8. TIME-OF-DAY TIMING PARAMETERS

SHA traffic signal controllers are capable of implementing different detection and phase settings for various times of day through the time-of-day (TOD) programs. These functions can be programmed for the entire intersection or for specific phases. The TOD timing parameters, as depicted in the SHA Signal Timing Chart, are shown in Figure 8.1.

![SHA Signal Timing Chart](image)

**Figure 8.1 – Time-of-day timing parameters on SHA Signal Timing Chart**

A. Common TOD Applications

The following TOD program functions are commonly used by SHA. Additional information and programming instructions can be found in Econolite’s Advanced System Controller ASC/2 Family Programming Manual.

**Alternate Vehicle Extension – ALT VEH EXTSN**

The alternate vehicle extension enables the use of the VEH EXT2 (vehicle extension interval) as the preset gap. This feature is commonly used along approaches with a high volume of school buses because of the slower acceleration rates. A slightly higher vehicle extension interval is used during specific time periods to prevent premature “gap outs” for the phase(s) on that approach.
**Maximum Green Enable – MAX2 ENABLE and MAX3 ENABLE**

The maximum green enable functions, MAX2 ENABLE and MAX3 ENABLE, set MAX II or MAX III as the maximum green interval for a specific phase, respectively. These parameters are often used at isolated intersections where demand varies during various times of day or where specific maximum green intervals are required for special events or incident management.

**B. Additional TOD Applications**

Although less commonly used in the field, the following TOD functions can be implemented using SHA’s controllers. Additional information and programming instructions can be found in Econolite’s *Advanced System Controller ASC/2 Family Programming Manual*.

**Alternate Phase Sequence – ALTERNATE SEQUENCE**

This function alters the signal phase sequence by reversing the order in which conflicting phases are serviced. An appropriate application for the ALTERNATE SEQUENCE function is providing lead and/or lag left-turn phasing or reversing the sequence of split phases during various times of day.

**Conditional Service Inhibit – COND SERV INH**

The selection of the conditional service inhibit function allows phases to be re-serviced (lead plus lag) within the same cycle.


The selection of these functions sets the recall switch in the controller to return service to a particular phase even if demand is not present.
9. SYSTEM CONTROL

A traffic signal system consists of two or more coordinated, signalized intersections, which may be along an arterial or throughout a grid of streets. Signal coordination is provided by means of a consistent time relationship for operations at each of the intersections within the system called the offset.

The primary objective of a coordinated signal system is to provide continuous movement and minimize delay throughout the signal system. This requires selecting, implementing, and monitoring signal timing patterns that address the capacity needs at each of the individual intersections as well as the combined needs of the entire signal system. Volume trends can be identified by performing volume studies on various approaches (e.g., turning movement counts and ATR counts). Typically, patterns are developed for the following periods:

- AM peak
- Mid-day
- PM peak
- Night
- Weekend
- Special (e.g., schools, shift work, sporting events, etc.)

The core settings for signal timing patterns are cycle length, split, and offset. System signal timing parameters are depicted in the SHA Signal Timing Chart as presented in Figure 9.1.
### STD FORMAT

<table>
<thead>
<tr>
<th>COORD PATTERN</th>
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<td>CYCLE LENGTH</td>
<td>C/O/S</td>
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### PLAN FORMAT

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<td>PHASE OMIT</td>
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<tr>
<td>SPARE</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td>F</td>
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</tbody>
</table>

**Figure 9.1 – Signal system timing parameters in the SHA Signal Timing Chart**
A. Cycle Length

The cycle length is the amount of time needed for the traffic signal controller to complete one sequence of all of the signal phases, including the yellow change and red clearance intervals. In order for a group of traffic signals to operate as a coordinated signal system, all intersections typically operate with the same fixed cycle length – i.e., system cycle length.

The system cycle length is usually equal to the required cycle length for the intersection with the highest total intersection critical lane volume (CLV) – i.e., the “critical intersection.” The system cycle length is calculated for each signal timing pattern or for various time periods. There are several ways to determine the optimum cycle length, such as critical lane volume analysis, Webster’s formula, or signal timing optimization software.

Table 9.1 presents SHA’s recommended maximum cycle lengths for various levels of service, or total intersection critical lane volume, based on the number of conflicting movements.

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>2 Conflicting Movements</th>
<th>3 Conflicting Movements</th>
<th>4 Conflicting Movements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2 phases</td>
<td>3 – 5 phases</td>
<td>6 – 8 phases</td>
</tr>
<tr>
<td>A</td>
<td>CLV &lt; 1,000 vph</td>
<td>90</td>
<td>100</td>
</tr>
<tr>
<td>B</td>
<td>1,000 ≤ CLV ≤ 1,150 vph</td>
<td>90</td>
<td>100</td>
</tr>
<tr>
<td>C</td>
<td>1,151 ≤ CLV ≤ 1,300 vph</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>D</td>
<td>1,301 ≤ CLV ≤ 1,450 vph</td>
<td>120</td>
<td>135</td>
</tr>
<tr>
<td>E</td>
<td>1,451 ≤ CLV ≤ 1,600 vph</td>
<td>135</td>
<td>150</td>
</tr>
<tr>
<td>F</td>
<td>CLV &gt; 1,600 vph</td>
<td>150</td>
<td>165</td>
</tr>
</tbody>
</table>

Webster’s Formula

Determining the “optimal” cycle length requires balancing two competing characteristics – capacity and delays/queues. With shorter cycle lengths, a larger percentage of the cycle is consumed by the yellow change and red clearance intervals. Additionally, a higher percentage of the cycle length is occupied by start-up (lost) time for each movement with shorter cycles. With longer cycle lengths, there is more effective green time per hour, but longer cycles can result in longer delays and queues.

An alternative method for determining the background cycle length is Webster’s formula, which minimizes intersection delay based on total lost time (i.e., change and clearance intervals) and critical lane volume.
B. Split

The split is the amount of time, expressed in seconds or percentage of the total cycle length, allocated to each phase in the signal sequence. The split for each phase includes the green, yellow change, and red clearance intervals. As shown in Figure 9.2, the sum of the splits for the critical phases at an intersection equals the cycle length in seconds (or 100 percent).

![Figure 9.2 – Cycle length and split]

The following should be considered when determining splits:

- Splits should not be arbitrarily increased or decreased. With a fixed system cycle length, increasing the split for one phase requires decreasing the split for a conflicting phase—i.e., improving one movement potentially degrades another movement.
- Splits must be long enough to accommodate the sum of the minimum green interval (MIN...
GREEN), yellow change, and red clearance intervals.  
- Splits for phases with concurrent vehicular and pedestrian movements should be long enough to accommodate the sum of the pedestrian walk, pedestrian clearance (flashing don’t walk), yellow change, and red clearance intervals.
  - If pedestrian actuations are infrequent, the split should be calculated based on vehicular demand, which often requires less time than the pedestrian intervals. At these locations, a pedestrian actuation will cause the traffic signal controller to override the programmed split in order to service the programmed pedestrian timing intervals. This will cause the controller to “borrow” split timing from subsequent phases in the phasing sequence to ensure the pedestrian movement receives the programmed timing intervals. This override could potentially cause the intersection to lose coordination.
  - If pedestrian actuations are frequent, the split should be based on the required pedestrian times to reduce coordination impacts.

Splits can be determined manually or through the use of traffic signal timing optimization software. This section presents three different CLV-based methods for manually determining the split for each signal phase:

1. Proportional V/C ratio method
2. Greenshields’ green time method
3. Poisson distribution method
Method 1: Proportional Volume-to-Capacity (V/C) Ratio Method

This approach apportions the cycle length based on the ratio, or percentage, of each phase’s critical lane volume and the total intersection critical lane volume. This is the most common methodology for determining splits and often the most effective way to equitably service all of the movements at an intersection.

Calculating splits: Proportional V/C Ratio Method

1. Using turning movement counts, determine the highest 60-minute volume for each phase. **NOTE:** The volumes for each movement do not need to be from the same 60-minute period. However, the 60-minute periods should be from the same overall time period during which the corresponding signal timing pattern will operate.
2. Calculate the critical lane volume for each phase.
3. Calculate the total intersection critical lane volume and identify the critical phases.
4. From Table 9.1, or using Webster’s formula, determine the cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases. **NOTE:** If the intersection is not the “critical intersection,” skip this step and use the system cycle length for the remaining steps.
5. Sum the yellow change intervals for the critical phases, \( Y_{TOTAL} \).
6. Sum the red clearance intervals for the critical phases, \( R_{TOTAL} \).
7. Calculate the total available green time (per cycle), \( G_{TOTAL} \), using the background cycle length (Step 4), \( Y_{TOTAL} \) (Step 5), and \( R_{TOTAL} \) (Step 6).

\[
G_{TOTAL} = \text{Cycle length} - Y_{TOTAL} - R_{TOTAL}
\]

8. For each phase, apportion the total available green time (per cycle), \( G_{TOTAL} \) (Step 7), based on the ratio of critical lane volume for each phase (Step 2) and the total intersection critical lane volume (Step 3).

\[
G_{PHASE} = \frac{G_{TOTAL} \times \text{Phase CLV}}{\text{Total intersection CLV}}
\]

9. For each phase, sum the green (Step 8), yellow change, and red clearance intervals.

\[
\text{Split} = G_{PHASE} + Y_{PHASE} + R_{PHASE}
\]
Example 9.1: Determine the split for each phase using the proportional V/C ratio method given the following information:

- Phasing sequence – exclusive left-turn phasing (major street) and split side-street phasing
- Lane use factor (LUF) for double left-turn lanes: 0.60 (see Chapter 11)
- Change and clearance intervals:
  - Phase 1 & Phase 5 yellow change intervals: 3 sec
  - Phase 2 & Phase 6 yellow change intervals: 5 sec
  - Phase 3 & Phase 4 yellow change intervals: 4 sec
  - Red clearance intervals (all phases): 2 sec

Step 1: Using turning movement counts, determine the highest 60-minute volume for each phase.

See given turning movement volumes.

Step 2: Calculate the critical lane volume for each phase.
- Phase 1 (NB left): CLV = 450 × 0.60 = 270 vph
- Phase 2 (SB through): CLV = 515 vph
- Phase 3 (WB left & through): CLV = max(150, 300) = 300 vph
- Phase 4 (EB left & through): CLV = max(215, 375) = 375 vph
- Phase 5 (SB left): CLV = 350 × 0.60 = 210 vph
- Phase 6 (NB through): CLV = 500 vph

Step 3: Calculate the total intersection critical lane volume.
- (NB left + SB through): CLV = 270 + 515 = 785 vph \(^{critical}\)
- (SB left + NB through): CLV = 210 + 500 = 710 vph
- (WB left & through): CLV = 300 vph \(^{critical}\)
- (EB left & through): CLV = 375 vph \(^{critical}\)
- Total intersection critical lane volume = 785 + 300 + 375 = 1,460 vph (LOS E)
Step 4: From Table 9.1, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.

*From Table 9.1 (1,460 vph, 4 conflicting movements) → 165-sec cycle length*

Step 5: Sum the yellow change intervals for the critical phases, $Y_{TOTAL}$.

*From Step 3, the critical phases are Phases 1, 2, 3, & 4.\n\n$Y_{TOTAL} = 3 + 5 + 4 + 4 = 16$ sec*

Step 6: Sum the red clearance intervals for the critical phases, $R_{TOTAL}$.

*From Step 3, the critical phases are Phases 1, 2, 3, & 4.\n\n$R_{TOTAL} = 2 + 2 + 2 + 2 = 8$ sec*

Step 7: Calculate the total available green time (per cycle), $G_{TOTAL}$, using the background cycle length (Step 4), $Y_{TOTAL}$ (Step 5), and $R_{TOTAL}$ (Step 6).

$G_{TOTAL} = 165 - 16 - 8 = 141$ sec

Step 8: For each phase, apportion the total available green time (per cycle), $G_{TOTAL}$ (Step 7), based on the ratio of critical lane volume for each phase (Step 2) and the total intersection critical lane volume (Step 3).

Phase 1 (NB left): 141 sec $\times \frac{270 \text{ vph}}{1,460 \text{ vph}} = 26.1$ sec
Phase 2 (SB through): 141 sec $\times \frac{515 \text{ vph}}{1,460 \text{ vph}} = 49.7$ sec
Phase 3 (WB left & through): 141 sec $\times \frac{300 \text{ vph}}{1,460 \text{ vph}} = 29.0$ sec
Phase 4 (EB left & through): 141 sec $\times \frac{375 \text{ vph}}{1,460 \text{ vph}} = 36.2$ sec
Phase 5 (SB left): 141 sec $\times \frac{210 \text{ vph}}{1,460 \text{ vph}} = 20.3$ sec
Phase 6 (NB through): 141 sec $\times \frac{500 \text{ vph}}{1,460 \text{ vph}} = 48.3$ sec

Step 9: For each phase, sum the green (Step 8), yellow change, and red clearance intervals.

Phase 1 (NB left): 26 + 3 + 2 = 31 sec *critical
Phase 2 (SB through): 50 + 5 + 2 = 57 sec *critical
Phase 3 (WB left & through): 29 + 4 + 2 = 35 sec *critical
Phase 4 (EB left & through): 36 + 4 + 2 = 42 sec *critical
Phase 5 (SB left): 20 + 3 + 2 = 25 sec
Phase 6 (NB through): 48 + 5 + 2 = 55 sec

Sum of critical phases’ splits: 31 + 57 + 35 + 42 = 165 sec (cycle length)
Method 2: Greenshields' Green Time (Vehicle Headway) Method

This approach calculates the split by determining the amount of green time required to serve the number of “critical” vehicles arriving each cycle for each phase. The green time requirements, presented in Table 9.2, are based on Bruce Greenshields’ research on vehicle headways and saturation flow rates at intersections.

Table 9.2 – Green time requirements (Greenshields’ vehicle headway model)

<table>
<thead>
<tr>
<th>Avg. Vehicles per Cycle per Lane</th>
<th>Green Time per Vehicle</th>
<th>Cumulative Green Time</th>
<th>Avg. Vehicles per Cycle per Lane</th>
<th>Green Time per Vehicle</th>
<th>Cumulative Green Time</th>
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</table>
Calculating splits: Greenshields’ Method

1. Using turning movement counts, determine the highest 60-minute volume for each phase. 
   NOTE: The volumes for each movement do not need to be from the same 60-minute period. 
   However, the 60-minute periods should be from the same overall time period during which 
   the corresponding signal timing pattern will operate.

2. Calculate the critical lane volume for each phase.

3. Calculate the total intersection critical lane volume.

4. From Table 9.1, or using Webster’s formula, determine the background cycle length using 
   the total intersection critical lane volume (Step 3) and the number of conflicting 
   movements/number of phases. NOTE: If the intersection is not the “critical intersection,” 
   skip this step and use the system cycle length for the remaining steps.

5. Calculate the average number of vehicles per cycle (per lane) for each phase using the critical 
   lane volume (Step 2) and the background cycle length (Step 4).

\[
\text{Average vehicles per cycle (per lane) = \frac{\text{Critical lane volume} \times \text{Background cycle length}}{3,600 \text{ sec/hr}}}
\]

6. Using Table 9.2, determine the cumulative green time required to serve the average vehicles 
   per cycle for each phase (Step 5).

7. For each phase, sum the green (Step 6), yellow change, and red clearance intervals.

\[
\text{Split} = G_{\text{phase}} + Y_{\text{phase}} + R_{\text{phase}}
\]

8. Sum the splits for the critical phases.
   a. If the cycle length is greater than the sum of the splits for the critical phases (i.e., under capacity):
      i. The excess time is typically allocated to the coordinated phases (i.e., phases 2 and 6) to increase the greenband.
      ii. A small portion of the excess time should be allocated to the minor street 
          movements to prevent the phases from frequently “maxing out.”
   b. If the cycle length is less than the sum of the splits for the critical phases (i.e., over capacity):
      i. Increase the cycle length and repeat Steps 5 through 8. This option may not 
         be applicable if a specific system cycle length must be maintained.
      ii. Consider using Method 1: Proportional V/C Ratio Method to allocate the 
          splits based on the proportion of each phase’s critical lane volume and the 
          total intersection critical lane volume.
**Example 9.2:** Determine the split for each phase using Greenshields’ method given the following information:

- **Phasing sequence** – exclusive left-turn phasing on all approaches
- **Lane use factor (LUF) for double left-turn lanes:** 0.60 (see Chapter 11)
- **Change and clearance intervals:**
  - Left-turn phase (phases 1, 3, 5, and 7) yellow change intervals: 3 sec
  - Through phase (phases 2, 4, 6, and 8) yellow change intervals: 4 sec
  - Red clearance intervals (all phases): 2 sec

**Step 1:** Using turning movement counts, determine the highest 60-minute volume for each phase.

*See given turning movement volumes.*

**Step 2:** Calculate the critical lane volume for each phase.

- **Phase 1 (NB left):** CLV = 450 × 0.60 = 270 vph
- **Phase 2 (SB through):** CLV = 515 vph
- **Phase 3 (EB left):** CLV = 220 vph
- **Phase 4 (WB through):** CLV = 300 vph
- **Phase 5 (SB left):** CLV = 350 × 0.60 = 210 vph
- **Phase 6 (NB through):** CLV = 500 vph
- **Phase 7 (WB left):** CLV = 100 vph
- **Phase 8 (EB through):** CLV = 375 vph
Step 3: Calculate the total intersection critical lane volume.
(NB left + SB through): CLV = 270 + 515 = 785 vph *critical
(SB left + NB through): CLV = 210 + 500 = 710 vph
(EB left + WB through): CLV = 220 + 300 = 520 vph *critical
(WB left + EB through): CLV = 100 + 375 = 475 vph
Total intersection critical lane volume = 785 + 520 = 1,305 vph (LOS D)

Step 4: From Table 9.1, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.
From Table 9.1 (1,305 vph, 4 conflicting movements) → 150-sec cycle length

Step 5: Calculate the average vehicles per cycle (per lane) for each phase using the critical lane volume (Step 2) and the background cycle length (Step 4).
Phase 1 (NB left): CLV = 270 vph × (150 sec ÷ 3600) = 11 veh/cycle
Phase 2 (SB through): CLV = 515 vph × (150 sec ÷ 3600) = 21 veh/cycle
Phase 3 (EB left): CLV = 220 vph × (150 sec ÷ 3600) = 9 veh/cycle
Phase 4 (WB through): CLV = 300 vph × (150 sec ÷ 3600) = 12 veh/cycle
Phase 5 (SB left): CLV = 210 vph × (150 sec ÷ 3600) = 9 veh/cycle
Phase 6 (NB through): CLV = 500 vph × (150 sec ÷ 3600) = 21 veh/cycle
Phase 7 (WB left): CLV = 100 vph × (150 sec ÷ 3600) = 4 veh/cycle
Phase 8 (EB through): CLV = 375 vph × (150 sec ÷ 3600) = 16 veh/cycle

Step 6: Using Table 9.2, determine the cumulative green time required to serve the average vehicles per cycle for each phase (Step 5).
Phase 1 (NB left): 11 veh/cycle → 26.8 sec
Phase 2 (SB through): 21 veh/cycle → 47.8 sec
Phase 3 (EB left): 9 veh/cycle → 22.6 sec
Phase 4 (WB through): 12 veh/cycle → 28.9 sec
Phase 5 (SB left): 9 veh/cycle → 22.6 sec
Phase 6 (NB through): 21 veh/cycle → 47.8 sec
Phase 7 (WB left): 4 veh/cycle → 12.0 sec
Phase 8 (EB through): 16 veh/cycle → 37.3 sec

Step 7: For each phase, sum the green (Step 6), yellow change, and red clearance intervals.
Phase 1 (NB left): 27 + 3 + 2 = 32 sec *critical
Phase 2 (SB through): 48 + 4 + 2 = 54 sec *critical
Phase 3 (EB left): 23 + 3 + 2 = 28 sec *critical
Phase 4 (WB through): 29 + 4 + 2 = 35 sec *critical
Phase 5 (SB left): 23 + 3 + 2 = 28 sec
Phase 6 (NB through): 48 + 4 + 2 = 54 sec
Phase 7 (WB left): 12 + 3 + 2 = 17 sec
Phase 8 (EB through): 37 + 4 + 2 = 43 sec

Step 8: Sum the splits for the critical phases.
Sum of critical phase splits: 32 + 54 + 28 + 35 = 149 sec < 150-sec cycle
Under capacity – Increase the Phase 2 split to 55 seconds.
Method 3: Poisson Distribution Method

The Poisson distribution method, similar to Greenshields’ method, is based on determining the amount of green time required to serve the number of “critical” vehicles arriving each cycle. Instead of using the average number of vehicles to determine the green time requirements and corresponding splits, this approach uses the Poisson distribution (i.e., random vehicle arrivals) to calculate the maximum (95th percentile) number of vehicles that will arrive each cycle. This methodology is more conservative than Greenshields’ method because it provides additional green time to account for fluctuations in demand. This method is commonly used at intersections that operate below capacity (i.e., where additional green time is available to service the “worst-case” queue) and at intersections where vehicle arrivals vary significantly.

Table 9.3 presents the maximum vehicles per cycle from the Poisson distribution and the corresponding green time requirements using Greenshields’ vehicle headway model.

<table>
<thead>
<tr>
<th>Avg. Vehicles per Cycle per Lane</th>
<th>Max No. of Vehicles per Cycle per Lane</th>
<th>Cumulative Green Time (Greenshields’)</th>
<th>Avg. Vehicles per Cycle per Lane</th>
<th>Max No. of Vehicles per Cycle per Lane</th>
<th>Cumulative Green Time (Greenshields’)</th>
</tr>
</thead>
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<td>14.1 – 14.9</td>
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<td>47.8</td>
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<td>15.0 – 15.7</td>
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</tr>
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<td>15.8 – 16.5</td>
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</table>
Calculating splits: Poisson Distribution Method

1. Using turning movement counts, determine the highest 60-minute volume for each phase. 
   NOTE: The volumes for each movement do not need to be from the same 60-minute period. However, the 60-minute periods should be from the same overall time period during which the corresponding signal timing pattern will operate.

2. Calculate the critical lane volume for each phase.

3. Calculate the total intersection critical lane volume.

4. From Table 9.1, or using Webster’s formula, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases. NOTE: If the intersection is not the “critical intersection,” skip this step and use the system cycle length for the remaining steps.

5. Calculate the average number of vehicles per cycle (per lane) for each phase using the critical lane volume (Step 2) and the background cycle length (Step 4).

\[
\text{Average vehicles per cycle (per lane)} = \frac{\text{Critical lane volume x Background cycle length}}{3,600 \text{ sec/hr}}
\]

6. Using Table 9.3, determine the maximum (95th percentile) number of vehicles per cycle for each phase and cumulative green time required to serve those vehicles.

7. For each phase, sum the green (Step 6), yellow change, and red clearance intervals.

\[\text{Split} = G_{\text{PHASE}} + Y_{\text{PHASE}} + R_{\text{PHASE}}\]

8. Sum the splits for the critical phases.
   a. If the cycle length is greater than the sum of the splits for the critical phases (i.e., under capacity):
      i. The excess time is typically allocated to the coordinated phases (i.e., phases 2 and 6) to increase the greenband.
      ii. A small portion of the excess time should be allocated to the minor street movements to prevent the phases from frequently “maxing out.”
   b. If the cycle length is less than the sum of the splits for the critical phases (i.e., over capacity):
      i. Increase the cycle length and repeat Steps 5 through 8. This option may not be applicable if a specific system cycle length must be maintained.
      ii. Consider using Method 1: Proportional V/C Ratio Method to allocate the splits based on the proportion of each phase’s critical lane volume and the total intersection critical lane volume.
Example 9.3: Determine the split for each phase using the Poisson distribution method given the following information:

- **Phasing sequence** – exclusive left-turn phasing on all approaches
- **Lane use factor (LUF) for double left-turn lanes**: 0.60 (see Chapter 11)
- **Change and clearance intervals**:
  - Left-turn phase (phases 1, 3, 5, and 7) yellow change intervals: 3 sec
  - Through phase (phases 2, 4, 6, and 8) yellow change intervals: 4 sec
  - Red clearance intervals (all phases): 2 sec

**Step 1:** Using turning movement counts, determine the highest 60-minute volume for each phase.

*See given turning movement volumes.*

**Step 2:** Calculate the critical lane volume for each phase.

- Phase 1 (NB left): $CLV = 390 \times 0.60 = 234 \text{ vph}$
- Phase 2 (SB through): $CLV = 420 \text{ vph}$
- Phase 3 (EB left): $CLV = 115 \text{ vph}$
- Phase 4 (WB through): $CLV = 150 \text{ vph}$
- Phase 5 (SB left): $CLV = 280 \times 0.60 = 168 \text{ vph}$
- Phase 6 (NB through): $CLV = 430 \text{ vph}$
- Phase 7 (WB left): $CLV = 80 \text{ vph}$
- Phase 8 (EB through): $CLV = 175 \text{ vph}$

**Step 3:** Calculate the total intersection critical lane volume.
Step 4: From Table 9.1, determine the background cycle length using the total intersection critical lane volume (Step 3) and the number of conflicting movements/number of phases.

From Table 9.1 (919 vph, 4 conflicting movements) \( \rightarrow \) 120-sec background cycle length

Step 5: Calculate the average vehicles per cycle (per lane) for each phase using the critical lane volume (Step 2) and the background cycle length (Step 4).

Phase 1 (NB left): \( \text{CLV} = 234 \text{ vph} \times \frac{120 \text{ sec}}{3600} = 7.8 \text{ veh/cycle} \)
Phase 2 (SB through): \( \text{CLV} = 420 \text{ vph} \times \frac{120 \text{ sec}}{3600} = 14.0 \text{ veh/cycle} \)
Phase 3 (EB left): \( \text{CLV} = 115 \text{ vph} \times \frac{120 \text{ sec}}{3600} = 3.8 \text{ veh/cycle} \)
Phase 4 (WB through): \( \text{CLV} = 150 \text{ vph} \times \frac{120 \text{ sec}}{3600} = 5.0 \text{ veh/cycle} \)
Phase 5 (SB left): \( \text{CLV} = 168 \text{ vph} \times \frac{120 \text{ sec}}{3600} = 5.6 \text{ veh/cycle} \)
Phase 6 (NB through): \( \text{CLV} = 430 \text{ vph} \times \frac{120 \text{ sec}}{3600} = 14.3 \text{ veh/cycle} \)
Phase 7 (WB left): \( \text{CLV} = 80 \text{ vph} \times \frac{120 \text{ sec}}{3600} = 2.7 \text{ veh/cycle} \)
Phase 8 (EB through): \( \text{CLV} = 175 \text{ vph} \times \frac{120 \text{ sec}}{3600} = 5.8 \text{ veh/cycle} \)

Step 6: Using Table 9.3, determine the maximum (95th percentile) number of vehicles per cycle for each phase and cumulative green time required to serve those vehicles.

Phase 1 (NB left): 7.8 veh/cycle \( \rightarrow \) 13 max veh/cycle \( \rightarrow \) 31.0 sec
Phase 2 (SB through): 14.0 veh/cycle \( \rightarrow \) 20 max veh/cycle \( \rightarrow \) 45.7 sec
Phase 3 (EB left): 3.8 veh/cycle \( \rightarrow \) 7 max veh/cycle \( \rightarrow \) 18.4 sec
Phase 4 (WB through): 5.0 veh/cycle \( \rightarrow \) 9 max veh/cycle \( \rightarrow \) 22.6 sec
Phase 5 (SB left): 5.6 veh/cycle \( \rightarrow \) 10 max veh/cycle \( \rightarrow \) 24.7 sec
Phase 6 (NB through): 14.3 veh/cycle \( \rightarrow \) 21 max veh/cycle \( \rightarrow \) 47.8 sec
Phase 7 (WB left): 2.7 veh/cycle \( \rightarrow \) 6 max veh/cycle \( \rightarrow \) 16.3 sec
Phase 8 (EB through): 5.8 veh/cycle \( \rightarrow \) 10 max veh/cycle \( \rightarrow \) 24.7 sec

Step 7: For each phase, sum the green (Step 6), yellow change, and red clearance intervals.

Phase 1 (NB left): 31 + 3 + 2 = 36 sec \( * \text{critical} \)
Phase 2 (SB through): 45 + 4 + 2 = 51 sec \( * \text{critical} \)
Phase 3 (EB left): 18 + 3 + 2 = 23 sec \( * \text{critical} \)
Phase 4 (WB through): 23 + 4 + 2 = 29 sec \( * \text{critical} \)
Phase 5 (SB left): 25 + 3 + 2 = 30 sec
Phase 6 (NB through): 48 + 4 + 2 = 54 sec
Phase 7 (WB left): 16 + 3 + 2 = 21 sec
Phase 8 (EB through): 25 + 4 + 2 = 31 sec

Step 8: Sum the splits for the critical phases.

Sum of critical phase splits: 36 + 51 + 23 + 29 = 139 sec > 120-sec cycle

Over capacity – Consider increasing the cycle length or using Method 1: Proportional V/C Ratio Method.
C. Offset

One of the primary goals of signal system timing is to progress the majority of the motorists traveling between interconnected signals, or along a signalized corridor, through the intersections without stopping. This is accomplished by using an offset, which is the difference between the start of the main street (or coordinated phase) green interval at one intersection and the start of the main street (or coordinated phase) green interval at another intersection. The offset can be expressed in seconds; however, SHA’s preferred application is to reference the offset as a percentage of the cycle length.

As depicted in Figure 9.3, one intersection in the system is usually designated as the master and all intersection offsets are referenced to the master intersection. The intersection with the highest critical lane volume (i.e., the “critical intersection”) is often the master intersection because this intersection usually has the smallest greenband potential. The intersection of two signal systems is also designated as the master intersection so that both arterials can be coordinated using a common reference point.

![Figure 9.3 – Offsets](image-url)
Calculating offsets as a percentage of the cycle length

1. Identify the time during the cycle when the green interval begins for the coordinated phase at the master intersection.
2. Identify the time during the cycle when the green interval begins for the coordinated phase at the upstream/downstream intersection.
3. Calculate the time difference (absolute value) between the master intersection and the upstream/downstream intersection.
4. Time offset (seconds) = |Intersection start of green – master intersection start of green|
5. Divide the time offset by the cycle length.

\[
\text{Offset (percent of cycle length)} = \frac{\text{Time offset (seconds)}}{\text{Cycle length (seconds)}}
\]

Time-Space Diagrams

Offsets are calculated based on the amount of time it takes to travel the distance (or space) between adjacent signalized intersections. Offsets are typically determined through the development of a time-space diagram, which can be drawn manually or developed using signal timing software.

A time-space diagram is a plot of distance versus time that is used to visualize the relationship between signal system timing parameters and to determine the effects on system wide coordination when parameters are modified. The location of each signalized intersection, and the corresponding spacing between each intersection (in feet), is plotted on the vertical axis (Y axis). The horizontal axis (X axis) represents time and it is used to depict the cycle length, offsets, and the splits for the coordinated phase at each intersection. Time is typically referenced to the beginning of green for the coordinated phase at the master intersection (i.e., X equals zero at the master intersection). The slope of any line on the time-space diagram represents travel speed, so the speed limit can also be depicted on the diagram. Major street left-turn phases are typically depicted on the diagram using a hatched area. A sample time-space diagram is provided in Figure 9.4.
Figure 9.4 – Time-space diagram
The primary output from a time-space diagram is greenband (or bandwidth). Greenband is the amount of time, in seconds, that a specific movement is progressed through a series of signalized intersections without stopping. The greenband can extend through all of the signals within a system (i.e., “system” greenband) or it can extend through only a portion of the signals within a system (i.e., “local” greenband). Both conditions are illustrated in Figure 9.5.

![Figure 9.5 – Greenband on time-space diagram](image)

Although the fundamental goal of signal system timing is to typically maximize the greenband for the coordinated movement(s), other traffic conditions and impacts should also be considered when developing timing patterns, such as adjusting timing parameters to avoid queue spillback (i.e., queue management), lowering cycle lengths to reduce delays, and modifying offsets to reduce the number of stops.
Early Green

One of the consequences of semi-actuated, coordinated signal systems is an early green, which occurs when the coordinated phase begins before its predetermined offset because a minor phase was skipped or terminated before it reached its maximum split (i.e., “gapped out”). Although an early green can reduce delays locally at an intersection, motorists who depart early may be required to stop at a downstream intersection if they arrive before the beginning of green for the coordinated phase. This can cause multiple stops for vehicles and a perception of poor signal coordination. The early green concept is illustrated in Figure 9.6.

![Figure 9.6 – Early green](image)

Because early greens can affect progression and cause unexpected stops along coordinated signal systems, accurately timing the splits for the minor (non-coordinated) movements is particularly important to avoid “gap outs” and correspondingly reduce the frequency of early greens. Another technique to avoid early greens is to start the coordinated phase earlier at the downstream intersection (i.e., shift the offset to the left); however, this may adversely affect progression in the opposite direction or reduce the greenband when the actuated phases “max out.”
One-way Progression

Signal systems along one-way roadways and two-way systems that are timed to provide progression exclusively in one direction are the simplest types of systems to coordinate. Determining the offsets for one-way systems is based on the time it takes for motorists to travel between intersections at the posted speed limit.

Calculating offsets for one-way progression

1. Measure the distance between the master intersection and the upstream/downstream intersection.
2. Divide the distance by the posted speed limit to calculate the time offset.

\[
\text{Time offset (seconds)} = \frac{\text{Distance between intersection and master intersection (feet)}}{1.47 \times \text{Posted speed limit (MPH)}}
\]

where,

1.47 = conversion constant for MPH to ft/sec

3. Divide the time offset by the cycle length.

\[
\text{Offset (percent of cycle length)} = \frac{\text{Time offset (seconds)}}{\text{Cycle length (seconds)}}
\]

Figure 9.7 illustrates the concept of one-way progression, where the beginning of green for the coordinated phase is solely based on travel time and the system wide greenband is equal to shortest split for the coordinated phase.
When traffic is moving in both directions along the same street, progressing both through movements can be much more complicated. Because both directions of travel usually receive green indications at the same time (i.e., the coordinated phases begin simultaneously), shifting offsets to favor progression in one direction can negatively impact progression in the other direction.

When traffic volumes along the two-way street are higher in one direction versus the other during various times of day, the time-space diagram should be prepared to favor progression in the primary direction; however, consideration should also be given to the opposing direction to help reduce queues, delays, and stops by identifying offsets that are acceptable for traffic in both directions.

There are several “classic” offset patterns available to provide two-way progression. These offset patterns are based on basic signal timing assumptions; however, once the generic offset pattern is developed, the splits and offsets at individual intersections can be refined as necessary.
**Single alternate pattern (Half-cycle offset)**

The *single alternate pattern* is a timing plan where the offset from one intersection to the next is equal to half the cycle length (i.e., the offsets alternate between 0 and 50). Therefore, the green interval for the coordinated phase at one intersection begins at the end of the green interval at an adjacent intersection, as depicted in Figure 9.8. The single alternate pattern is typically used to provide progression in both directions for systems that operate with lower speed limits and shorter cycle lengths (e.g., a system with a 60-second cycle length, 30 MPH speed limit, and ¼ mile signal spacing).

![Figure 9.8 – Single alternate pattern](image)

**Double alternate pattern**

The *double alternate pattern* is a timing plan where the offsets at pairs of intersections alternate every half cycle with adjacent pairs of intersections (i.e., the “paired offsets” alternate between 0 and 50). Therefore, the green interval for the coordinated phase at the first pair of intersections begins at the end of the green interval at the adjacent pair of intersections, as shown in Figure 9.9. The double alternate pattern is typically used to provide progression in both directions for systems that operate with higher speed limits and longer cycle lengths (e.g., a system with a 120-second cycle length, 45 MPH speed limit, and 2,000-foot signal spacing).
Zero offset pattern (Simultaneous offset pattern)

A timing pattern in which the green intervals for the coordinated phases at all of the signals in the system begin at the same time is called a zero offset pattern. This offset pattern is occasionally used when the distance between signals is short (i.e., less than 500 feet) and two-way progression is desired. In long signal systems, the main street progression is eventually terminated (i.e., the system wide greenband is zero); however, a zero offset pattern can provide efficient “local” greenbands when minor street volumes are relatively low and the majority of the cycle can be assigned to the coordinated phases. Zero offset patterns are also useful in congested urban networks where simultaneous green intervals allow queues to clear between intersections before platoons arrive. One common problem associated with this pattern is that it tends to encourage speeding when motorists try to “beat the red light.”
D. Additional Coordination Tools

Lead/Lag Left-turn Phasing

At intersections where only exclusive left-turn phases are provided along the coordinated street, lead/lag left-turn phasing can be used to improve two-way progression and reduce queues and delays for left-turning motorists by serving the left-turn phases when platoons arrive from upstream intersections. Specifically, lead/lag phasing can increase the greenband for both directions because the left-turn movement and adjoining through movement can be served simultaneously instead of operating the opposing left-turn phases concurrently.

As shown in Figure 9.10, with lead/lag left-turn phasing, an exclusive lead left-turn phase operates first in one direction. Next, the through movements are served, and then an exclusive lag left-turn phase operates in the opposite direction. The benefits to two-way coordination are shown in Figure 9.11.

![Figure 9.10 - Phase sequence for lead/lag left-turn phasing](image-url)
Half Cycle Length (Double Cycling)

Instead of providing the same cycle length at all intersections within a signal system, minor intersections can operate with a half cycle length. Coordination is still maintained along the major street because the offsets at adjacent intersections are synchronized every other cycle, as shown in Figure 9.12. This phasing technique can significantly reduce queues and delays for minor movements at intersections with lower traffic volumes and/or less signal phases, such as signalized intersections at on-ramps and off-ramps. Half cycle lengths are also effective at intersections that are immediately adjacent to a signalized arterial (i.e., the first signal “off” of the signal system).
It is important to note that half cycle lengths are less effective when the signal operating with a half cycle length cannot accommodate the entire greenband or where there is a high frequency of early greens at upstream intersections. These conditions can increase stops and delays along the arterial.

**Modified Half Cycle Length (Uneven Double Cycle)**

A variation of the half cycle, the *modified half cycle length*, operates using a full cycle length; however, all phases are serviced twice per cycle using a custom ring structure with “uneven” splits. This type of phasing allows the intersection to serve the entire greenband to maintain progression along the major street and reduce queues and delays for the minor movements. Modified half cycle lengths are most effective along one-way arterials, at half signals (i.e., two-phase signals), and at signals immediately downstream of the critical intersection.

As shown in Figure 9.13, the modified half cycle length consists of a long major street phase equal to or greater than the greenband, then a minor street phase, then the major street phase is re-serviced for a short period, and then the minor street phase is re-serviced. The second (i.e., shorter) major street green interval can often be used to progress a heavy minor movement from an upstream intersection, such as a minor street left-turn phase.
E. System Types

Traffic signal systems range in size from two adjacent, coordinated intersections to hundreds of intersections being coordinated for network wide progression. Although there are many different combinations of traffic system timing equipment and techniques, signal systems generally fall into three basic types—relay based, non-interconnected, and telemetry.

Relay Based System (Master-Slave)

A relay based signal system usually consists of two intersections hardwired together with an interconnect cable, where one intersection operates as a “master” intersection and the other intersection operates as a “slave” intersection. The “master” intersection typically has a higher total critical lane volume (i.e., is more congested) and operates fully-actuated. During the major street green interval, the “master” intersection “holds” the “slave” intersection in major street green to provide coordination. Once the major street green interval at the “master” intersection terminates, the “hold” is released and both intersections serve the minor movements. The “hold” is re-applied every cycle when the major street green interval is displayed at the “master” intersection.

Non-Interconnected System (NIC) (Time-based Coordinated)

Another basic type of traffic signal system is the non-interconnected (NIC) system, which consists of two or more signalized intersections coordinated by time (i.e., time-based coordinated). In order for the signals to remain coordinated, the time clock at each intersection must be set to exactly the same time. If the time clocks are off by even a few seconds, or if one intersection’s clock is faster or
slower than another intersection’s clock, the coordination will be inaccurate or “drift.” The time clocks are typically synchronized using a power company’s supplied frequency (if all of the signals have the same power source) or by using a GPS-based timing system. The NIC system operates with predetermined cycle lengths, splits, and offsets and the timing patterns can be varied by time-of-day and/or day-of-week timing plans.

The major advantage of an NIC system is that it is relatively inexpensive to install because of the absence of interconnection. The major disadvantage of the NIC system is that all signal timing adjustments and operational monitoring must be made in the field because it lacks two-way communication. Therefore, timing adjustments cannot be made remotely during unexpected incidents and events with a NIC system.

Telemetry System

The third basic type of traffic signal system is the telemetry system, which consists of two or more intersections interconnected by a two-way communication system. The interconnection can be provided using a hardwire cable (e.g., copper, fiber optic cable, coaxial cable, etc.) or interconnection can be achieved wirelessly (e.g., spread spectrum radio, microwave, etc.). Coordination in a telemetry system is controlled by either an on-street master controller or a central-based computer, and system sampling detection is typically provided to monitor traffic conditions throughout the system and to change signal timings accordingly.

Although installing a telemetry system can be much more expensive than a NIC system, a telemetry system can detect, and adjust signal timings for, changes in traffic conditions instead of relying solely on predetermined time-of-day signal timings. The telemetry system also allows for remote malfunction and signal timing adjustments instead of having to monitor operations and implement timings in the field. These features are extremely important where unexpected incidents and events, such as major freeway incidents, often occur because of the need to make immediate signal timing changes and to monitor system operation.

F. Advanced System Operations

Traffic Responsive

The most common method for implementing signal timing patterns is through time-of-day and day-of-week schedules, which change timing patterns at specific, predetermined time periods (e.g., AM, mid-day, and PM peak periods). When traffic conditions vary significantly within these time periods, or when traffic is inconsistent on a day-to-day basis, time-of-day schedules are not as effective in reducing delays and queues. Traffic responsive operation is an alternative to time-of-day control, which uses vehicle detection to monitor real-time traffic conditions – volume (or headways) and density (or spacing) – to select a timing pattern that is best suited for the current traffic conditions.

Traffic responsive systems use system detectors to monitor and control the signal system. The system detectors are capable of categorizing data based on the following functions:

- Congestion level – cycle length selection
- Major versus minor-street demand – split selection
- Arterial travel direction (inbound, outbound) – offset selection
Real-time traffic data is collected by system detectors, analyzed, and then compared to predefined data profiles for timing patterns with unique cycle lengths, splits, and offsets. When a profile match is identified, the corresponding timing pattern is implemented.

Because traffic responsive systems are only capable of selecting timing parameters not modifying them, this type of operation still requires transportation professionals to determine timing patterns and develop a “library” that is best suited for varying traffic conditions. Table 9.4 presents a sample traffic responsive matrix used to select timing patterns based on inbound, outbound, and average traffic volumes collected by the system detectors.

<table>
<thead>
<tr>
<th>Directional Volume</th>
<th>Arterial Volume/Congestion Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cycle Length / Split / Offset</td>
</tr>
<tr>
<td></td>
<td>Level 1</td>
</tr>
<tr>
<td>Inbound</td>
<td>Free</td>
</tr>
<tr>
<td>Outbound</td>
<td>Free</td>
</tr>
<tr>
<td>Average</td>
<td>Free</td>
</tr>
<tr>
<td>Non-arterial</td>
<td>Free</td>
</tr>
</tbody>
</table>

There is a transition period along a signal system when timing plans change, and coordination is temporarily interrupted as the cycle length and/or offsets are adjusted. Therefore, the “sensitivity” of a traffic responsive system should be considered when developing a selection matrix to avoid frequently changing timing plans.

**On-Street Master**

The primary function of an on-street master is to select the timing pattern for the traffic responsive system. The on-street master monitors and processes volume and density data from various system detectors then informs the other coordinated signal controllers to change to a new timing pattern when the traffic conditions match the predetermined characteristics in the selection matrix.

**Traffic Adaptive**

Similar to traffic responsive operations, traffic adaptive signal systems use vehicle detection to adjust signal timings based on real-time traffic conditions. However, traffic adaptive systems modify cycle lengths, splits, and offsets instead of only selecting traffic patterns with predetermined timing parameters. These types of signal systems are occasionally used when traffic volumes fluctuate significantly during the peak periods on a day-to-day basis and where the unexpected incidents and special events severely impact signal operations.

Traffic adaptive systems use system detectors upstream and downstream of signals to monitor traffic conditions and collect data. The data is then processed using a computer algorithm that optimizes the signal timings, which are then automatically implemented in the field. These types of real-time signal adjustments can reduce delays and queues and improve coordination along signalized corridors.
Although traffic adaptive systems rely primarily on vehicle detection and computer software, transportation professionals are still required to identify specific signal timing constraints or thresholds (e.g., minimum and maximum splits and cycle lengths) and determine how sensitive and reactive the signal system is to slight fluctuations in traffic conditions. Inputs to the software can be used to accomplish different signal timing goals, such as queue management, maximizing the greenband, or minimizing total intersection delays.

**Adaptive Control Software Lite (ACS Lite)**

The ACS Lite software, developed by the Federal Highway Administration, is a type of traffic adaptive control designed to retrofit existing closed loop systems without the upgrade and maintenance costs required to implement “full-version” traffic adaptive systems. ACS Lite consolidates the traffic adaptive monitoring and data processing into the on-street master, which then communicates with other local controllers.

The primary difference in signal timing operations between ACS Lite and “full-version” traffic adaptive software is that ACS Lite only optimizes splits and offsets not cycle lengths. Additionally, the software monitors and analyzes traffic conditions on a cycle-by-cycle basis not second-by-second basis so the system is slightly less responsive to fluctuations in traffic conditions.
CHAPTER 10

Special Operations

Interim Edition
10. SPECIAL OPERATIONS

This chapter provides information for several advanced signal timing features that are occasionally used by SHA and that are primarily based on specific applications of the parameters presented in Chapters 6 and 9.

A. Advance Warning Flasher (AWF)

An advance warning flasher (AWF) is a specific type of hazard identification beacon that is installed upstream of a conventional traffic signal to warn approaching motorists prior to the onset of the yellow change interval. As shown in Figure 10.1, an electrically-controlled (flashing) RED SIGNAL AHEAD sign is commonly installed as part of the AWF assembly. AWF assemblies can also consist of SIGNAL AHEAD – PREPARE TO STOP WHEN FLASHING signs with yellow warning beacons.

Advance warning flashers can be used where the signal visibility does not meet the minimum sight distance requirements in Table 4D-1 of the Maryland Manual on Uniform Traffic Control Devices (MdMUTCD) or where a signalized intersection is “unexpected” (e.g., at an isolated intersection along a high-speed roadway or at the first signalized intersection downstream of an expressway).

Figure 10.1 – Advance Warning Flasher (US 50 and Maryland 213)

Prior to the onset of the yellow change interval, the AWF begins to alternately flash to warn approaching motorists of the pending phase termination. This overlap interval (i.e., the time between when the AWF begins to flash and the beginning of the yellow change interval) is the leading flash time or AWF overlap interval. The AWF flashes throughout the entire leading flash time and continues to flash while the corresponding phase receives a red indication. To further warn approaching motorists of the potential for queued vehicles along the approach, the AWF continues to flash for about 10 seconds after the start of the corresponding green interval.

The leading flash time is a type of clearance interval because it allows a motorist who crossed under the AWF immediately before it began to flash to enter the intersection on green. As shown, this interval is based on the distance between the AWF and the corresponding downstream stop line and the posted speed limit along the approach.
Determining the AWF leading flash time (AWF overlap interval)

The calculation for the AWF leading flash time is similar to the red clearance interval calculation in the Institute of Transportation Engineers (ITE) Traffic Engineering Handbook. The AWF formula is as follows:

\[
\text{AWF leading flash time} = \frac{L}{1.47 \times V}
\]

where,

- \( L \) = distance along the approach between the AWF assembly and the corresponding stop line (feet)
- \( V \) = posted speed limit on the approaching roadway (MPH)
- 1.47 = conversion factor for miles per hour to feet per second

Although advance warning flashers are beneficial to approaching vehicles, the corresponding overlap interval can adversely affect traffic operations on the opposing approach and minor street approaches. Specifically, the minimum green interval for the phase with the AWF must equal or exceed the leading flash time, which can cause excessive minor street delays during off-peak periods, and the overlap interval requires a predetermined phase termination (i.e., fixed yield point). As a result of the fixed yield point, the passage detection on the opposing approach cannot extend the phase during the overlap interval, which effectively eliminates the dilemma zone protection for vehicles on the opposing approach to an AWF. Further, the signal controller selects the next phase for service prior to the start of the overlap interval, which can potentially cause the subsequent minor street phase to be skipped if vehicles arrive during the leading flash time.

B. Coordinated-Actuated Systems

When a signal operates as part of a coordinated signal system, the coordinated phases (often the major street through phases) typically function in non-actuated mode. Although this type of operation allows for a predetermined yield point for progressing vehicle platoons, the non-actuated mode may unintentionally provide excess green time for the coordinated movements that could be utilized more effectively by a minor movement. Similarly, a coordinated phase might “max out” when demand is still present along the approach and the subsequent minor phases have relatively low demand.

To serve fluctuating demand for the coordinated movements more effectively, SHA occasionally programs a coordinated phase to operate in coordinated-actuated mode. As shown in Figure 10.2, this mode allows the coordinated phase to operate with a predetermined, or “guaranteed,” minimum split that can be extended each cycle based on actual traffic demand. During the actuated portion of the phase, the split is extended using passage detection and the corresponding vehicle extension interval.
Although infrequently used, coordinated-actuated operation can be utilized in SHA’s signal controllers using the coordinated phase split extension (SPLIT EXTENSION) timing parameter. The entire coordinated-actuated split should be calculated using the hourly demand and the methods described in Chapter 9 and then divided into “guaranteed” and actuated (extensible) portions based on specific field observations. The extensible portion commonly ranges between 20 to 30 percent of the total split.

C. Split Demand (Queue Detection)

SHA occasionally installs queue detectors at locations where queues can extend onto adjacent roadways and cause safety or capacity problems – e.g., at a signalized intersection located at the end of an expressway off-ramp, as shown in Figure 10.3. When a queue detector indicates continuous vehicle presence for a specified period of time, the split demand function in the signal controller will automatically change to a signal pattern that favors the phase(s) needing additional green time. The split demand pattern operates using the same cycle length as the preceding pattern with reduced green times for the conflicting signal phases; therefore, the phase with increased demand is temporarily “borrowing” time from one or more phases.
Similar to the delay setting for conventional presence detection, split demand queue detectors operate with the DEMAND CALL TIME setting, which requires a vehicle to be present on the detector for a predetermined period of time – typically ten seconds – before calling the split demand pattern. Because operating a specific signal pattern that reduces queues on one approach may consequently impact operations on another approach, the split demand pattern, assigned in the SPLIT DEMAND PATTERN SELECTION, will only operate for a certain number of cycles. This setting is defined using the DEMAND CYCLE COUNT parameter, which is commonly assigned a value of two cycles.

**D. Preemption**

The transfer of standard signal control to a special signal operation is called *preemption*. In particular, preemption is used to implement a specific phase or special sequence by responding to an external command such as an emergency vehicle or train. Because a train is more difficult to control and stop than an emergency vehicle and because the safety implications are significantly higher at highway-rail grade crossings, light rail and railroad preemption takes precedence over emergency vehicle preemption. The preemption signal timing parameters are depicted in the SHA Signal Timing Chart, as shown in Figure 10.4.
Preemption transfers the right-of-way to the direction of the approaching emergency vehicle or train; however, a green interval is not always provided immediately after the preemption call is received. Specific standards regarding the shortening and omission of signal timing intervals during preemption and the corresponding transition into and out of preemption are included in Section 4D.13 of the MdMUTCD. For example, a conflicting green interval can be shortened (typically no less than ten seconds) and a conflicting pedestrian walk interval can be omitted; however, the vehicular change and clearance intervals and pedestrian clearance interval cannot be shortened or omitted.

**Emergency Vehicles**

Emergency vehicle preemption is used to obtain a green indication or hold an existing green interval for an authorized emergency vehicle, such as an ambulance or fire truck. Equipped emergency vehicles have an emitter that broadcasts a light or infrared signal to a receiver located adjacent to the signal heads. These receivers are assigned to specific vehicular phases (HOLD PHASES) that receive or hold the green interval during preemption. Emergency vehicle preemption can also be provided via a physical connection between a fire station and adjacent traffic signal. At these locations, HOLD PHASES are assigned to the preemption push button.

For preemption along approaches with exclusive left-turn phasing, SHA typically provides green indications for the through movement and the adjacent left-turn movement. Along approaches with exclusive/permissive left-turn phasing, the corresponding through phase and opposing through phase...
typically receive green indications during preemption to avoid creating the “left-turn trap” (see Chapter 4).

Preemption calls during peak periods can adversely affect operations at a signal or along a signal system because the preempted signal may require multiple cycles to return to normal operations. As a result, two specific preemption parameters are available in SHA’s signal controllers to define how a signal returns to normal timing operations – the EXIT PHASES and EXIT MAX settings.

The EXIT PHASES parameter defines which phases are serviced after a specific preemption routine. For example, a preemption call that extends the green interval for the major street through phases is typically followed by the next phase in the sequence to avoid skipping phases. On the contrary, a preemption call on a minor street movement may cause excessive queues and delays along the major street; therefore, exiting to the major street phases may be appropriate even though it may result in skipped phases. In order to service the additional demand on non-preempted phases, the EXIT MAX parameter can be used to define an extended (or reduced) maximum green interval for the exit phases.

**Light Rail and Railroad Crossings**

The preemption of a traffic signal by an approaching train provides for the safe clearing of highway vehicles and pedestrians from the highway-rail grade crossing and it allows the train to cross safely without obstruction. In general, railroad preemption operates as follows:

- When an approaching train reaches a certain point on the track, a circuit is activated (or "shunted") and a signal is sent to the railroad control cabinet
- The railroad control cabinet is interconnected with the traffic signal control cabinet. When the railroad controller receives a message that a train is approaching, it sends a signal to the traffic signal controller to begin its preemption sequence. The railroad controller also activates the railroad warning devices (e.g., flashing lights and/or gates).
- When the traffic signal controller receives a preemption call, it begins a preemption sequence utilizing a special preemption mode, which is a separate phasing sequence (compared to normal signal phasing operations)

**Types of Railroad Preemption**

There are two common types of railroad preemption – simultaneous and advance.

- **Simultaneous preemption** occurs when the railroad warning devices and traffic signal are notified of an approaching train at the same time. The traffic signal begins its preemption sequence at the same time the railroad warning devices are activated.
- **Advance preemption** occurs when notification of an approaching train is forwarded to the traffic signal controller by railroad equipment for a predetermined period of time prior to activating the railroad warning devices.
Federal guidelines including the Manual on Uniform Traffic Control Devices (MUTCD) specify that active warning devices must operate for a minimum of 20 seconds prior to a train arriving at a crossing. This is known as the minimum time (MT). Any time in excess of the MT is defined as clearance time (CT). CT is additional time in excess of MT that may be provided by the railroad company to provide safe clearing of highway vehicles from the crossing. Buffer time (BT) is sometimes provided by the railroad company to account for variability in train speeds and other parameters. In some cases, equipment response time (ERT) is also required. ERT accounts for the time required for the railroad controller to respond to the circuit. The total warning time (TWT) provided by the railroad is equal to:

\[ TWT = MT + CT + BT + ERT \]

If more warning time is required by the traffic signal, additional time can be provided by the railroad in the form of advance preemption time (APT). APT is the amount of time that the traffic signal is notified and starts its preemption sequence prior to activating the railroad active warning devices. The total approach time (TAT) is the total time provided by the railroad equipment to the traffic signal controller to handle a preemption sequence and it is equal to:

\[ TAT = TWT + APT \]

When the traffic signal controller receives the preemption signal from the railroad controller, it begins a standard sequence of events. The goal of the preemption sequence is to clear all vehicles that may be queued between the stop line and the railroad tracks on the crossing street approach (see Figure 10.5) in the shortest time possible. This sequence is subdivided into two sections – right-of-way transfer time (RWTT) and queue clearance time.
RWTT is the maximum amount of time needed for the “worst-case” condition prior to displaying the track green phase interval. RWTT is the sum of the preemption verification and response time (the time required for traffic signal equipment to react to a preemption call) and the “worst-case” conflicting vehicle or pedestrian time. The “worst-case” conflicting vehicle time is composed of the minimum green time, yellow change interval, and red clearance interval. The “worst-case” conflicting pedestrian time is composed of the pedestrian clearance, yellow change, and red clearance intervals.

After the RWTT is complete, the queue must be cleared to avoid a potential vehicle-train conflict. The traffic signal phase required to clear the queue is designated as the track clearance phase (TRK CLR PHASE) and the time required to clear the queue is known as the queue clearance time. Figure 10.6 illustrates the typical distances that must be cleared. Essentially, the design vehicle must be moved through the design vehicle clearance distance (DVCD) and into the clear storage distance (CSD); however, before the design vehicle can start moving, the queue must be cleared in front of it.

The last component that must be taken into account in calculating the total preemption time is the desired minimum separation time. Desired minimum separation time acts as a buffer between when the queue is cleared and when the train arrives, providing a safety factor and giving comfort to both the train operator and the motorist. ITE recommended practice is to use four seconds for a desired minimum separation time.

The sum of the right-of-way transfer time, the queue clearance time, and the desired minimum separation time is the maximum preemption time (MPT). The railroad company should provide enough warning time to accommodate the MPT.
E. Single-Point Urban Interchange (SPUI)

A single-point urban interchange (SPUI), or single-point diamond interchange, is a type of diamond interchange that operates as a single signalized intersection instead of two closely-spaced, conventional intersections. As shown in Figure 10.7, the opposing left-turn movements on both the major street and off-ramps are aligned, which allows both off-ramps to operate concurrently as well as the major street left-turn phases. As a result, all of the movements at a SPUI are served by one three-phase signal. Because SPUI intersections are relatively large (i.e., require a significant amount of right-of-way and pavement to accommodate the skewed ramp alignments), SPUI signal phases often operate with long red clearance intervals as compared to conventional intersections.

Figure 10.7 – Single-point urban interchange (Maryland 170 and Maryland 100)

Section 4D.10 of the MdMUTCD states that the “red clearance interval should have a duration not exceeding 6 seconds,” and the sum of the yellow change and red clearance intervals generally should not exceed nine seconds. However, the red clearance intervals at single-point urban interchanges may exceed these guidelines because of the large intersection area and the long vehicle paths that are required to safely clear vehicles from the intersection. Red clearance intervals at SPUIs typically range from six to twelve seconds and the intervals should be calculated using the formula below with a "curb-to-curb" intersection width, \( W \), as depicted in Figure 10.8.

### Calculating the red clearance interval

The red clearance interval should be calculated using the following formula from the Institute of Transportation Engineers (ITE) *Traffic Engineering Handbook*:

\[
\text{Red clearance interval (RED CLEAR)} = \frac{W + L}{1.47 \times V}
\]

where,
- \( W \) = “curb-to-curb” width of the intersection (feet)
- \( L \) = length of vehicle, typically 20 feet
- \( V \) = posted speed limit on the approaching roadway (MPH)
- 1.47 = conversion constant for miles per hour to feet per second
F. Maryland “T” Intersection

As shown in Figure 10.9, a Maryland “T” intersection is a three-legged intersection with a physically-separated, inside (left) acceleration lane for the minor street left-turn movement. This intersection geometry allows the corresponding major street through movement to remain unsignalized and free-flowing.
Maryland “T” intersections have either exclusive left-turn phasing or flashing red arrow left-turn phasing along the free-flowing, major street approach. Maryland “T” intersections with exclusive phasing have conventional lead phasing (see Figure 10.10a) while intersections with a flashing red-arrow phase have lag left-turn phasing, as shown in Figure 10.10b.

The exclusive portion of the flashing red arrow left-turn phase is sequenced as a lag left-turn phase because left-turning vehicles may get “trapped” outside of the detection zone while attempting turn during the permissive (flashing red arrow) portion of the left-turn phase. When a call is placed on the minor street phase, a corresponding call is placed on the major street lag left-turn phase to ensure vehicles are not “trapped” in the turning path of conflicting minor street traffic.

Figure 10.10a – Phase sequence for Maryland “T” intersections with exclusive left-turn phasing

Figure 10.10b – Phase sequence for Maryland “T” intersections with flashing red arrow left-turn phasing
11. STANDARDS, FORMS, AND PROCEDURES

A. Roles and Responsibilities

MEMORANDUM

September 9, 1980

TO: Chief, Bureau of Traffic Engineering
Chief, Bureau of Traffic Operations
District Traffic Engineers

FROM: Thomas Hicks
Asst. Chief Engineer-Traffic

SUBJECT: Responsibilities – May 25th Memo

Attached is the May 25th memo as revised on September 9, 1980 taking into consideration your verbal and written comments of the draft presented this past June.

Please read it well and make copies available for all of your staff who will be affected by it. This directive forms a solid basis of understanding for all of our Traffic Division units and it is quite important that we adhere to it, regardless of our personal whims and wishes.

No traffic signal work will be approved or undertaken that has not come about through these procedures. I am directing the BTE and BTO Chiefs and the OEU to follow these and other approved procedures. If you have a question on the specific procedures of BTE, BTO, or OEU, please clarify the situation directly with them.

By separate memo, you will be receiving the Billwork procedures which were also discussed in June. It is intended to set forth the steps to be taken in handling billwork (new or modified) made necessary by new development.

Your cooperation in conforming to these procedures in your work will be appreciated.

Thomas Hicks

Attachment

cc: Mr. C. E. Buck
Mr. R. J. Bush
Mr. G. E. Cook
Mr. F. S. Jaworski
Mr. W. K. Lee, III
Mr. W. F. Lins, Jr.
Mr. A. L. Gardner
District Engineers
MEMORANDUM

TO: Chief, Bureau of Traffic Engineering
    Chief, Bureau of Traffic Operations
    District Traffic Engineers

FROM: Thomas Hicks
     Asst. Chief Engineer-Traffic

SUBJECT: Responsibilities

May 25, 1980
(Revised 9/9/80)

This is a revision of a similar directive dated May 25, 1973 and
is to restate the roles and responsibilities of the Bureaus of Traffic
Engineering and Traffic Operations and the District Traffic Engineers with
regard to traffic signal work. This summary is not a procedural directive
as individual procedures detailing tasks are established separately for
each TD unit. It does, however, establish overall responsibility with
respect to the roles of the individual TD units in the SHA traffic signal
program.

Points to consider and understand:

1. District Traffic Engineers handle all traffic signal STUDIES
to determine signalization (new or upgrading) needs.
   IMPORTANT - Studies for traffic signals are to be prepared for
   all proposed signal work along the state highway systems
   (i.e. state, developer, and county work) regardless of any
   agreements setting forth responsibility for design, construction,
   maintenance, and so forth. No signal work will be authorized
   that does not have a study, design request, and other supporting
   and design data as required here and in items 2 and 3 below.
   At this time, individual traffic signal projects may already
   appear on SHA program listings. If not, programming will be
   accomplished as noted in item 4.

2. DTE’s prepare the FUNCTIONAL DESIGN STUDY. This study is to
calculate all aspects of the proposed work, including consider-
ations relative to the total traffic operational needs of the
area. Among others, this includes the items of:

   Proposed signal phasing; special equipment (pre-emption,
   programmable heads, etc.); system needs; geometric changes
   (turning lanes, channelization, overlay work, etc.); and
   pavement markings, special signing, and pedestrian needs.
If geometric improvements are to be made at a later date, specific information is required now in the study report to permit a signalization layout at this time that will permit geometric construction later without disrupting signal operation and without requiring new equipment.

3. DTE’s shall prepare a STUDY REPORT in which the functional design needs are outlined and supported. The reports are to include the information noted in item #2 and are to be forwarded to the ACE-T along with the Design Request for signal design. A plan showing pavement markings, proposed geometrics, and unusual needs or conditions shall accompany the study report. The plan should show dimensions but need not be to scale. The DTE’s shall indicate in the study report any discussions and/or agreements with other jurisdictions regarding sharing or providing of costs, manpower, or equipment. See item §28.

4. The ACE-T, after approving the Design Request, forwards the file to the TD-Office Engineering Unit (OEU) for programming and/or recording. The OEU in turn forwards the request for design preparations to the BTE with copies to the OE and DTE.

5. BTE prepares the DETAILED LAYOUT CONSTRUCTION PLANS AND SPECIAL PROVISIONS for all signal projects.

6. BTE, in accomplishing item #5, is expected to communicate with both the DTE’s and BTO in the fulfillment of the design tasks assigned. The tasks, nevertheless, are BTE’s alone to accomplish. In doing so, they are to provide the functional operation as specified by the DTE without deviation.

7. Upon the completion of the Plans and Special Provisions, they are to be reviewed and approved by the DTE and the ACE-T following an approved procedure for F.I. and Final Reviews, and then forwarded to OEU for advertising.

8. With regard to contractor control, inspection, testing, acceptance, maintenance and operations, ALL SIGNAL PROJECTS ARE UNDER THE DIRECT CONTROL OF THE BUREAU OF TRAFFIC OPERATIONS AND NO ONE ELSE. Official maintenance and operations records are kept by BTO.
9. All appropriate Contractor comments, requests, and inquiries shall be directed (by the Engineer if formal District controlled projects) to BTO. Contractors shall NOT be approached by anyone other than the BTO for the purpose of doing work on traffic signal projects.

10. The directions expressed in item 9 shall also apply to all signal work not within formal or informal contracts. These may include matters pertaining to pole relocations on roadway construction projects, driveways or access construction points, miscellaneous electrical work, lighting work, or sign work. All construction contractor work in the Traffic Division shall be handled by BTO.

11. BTO will call upon BTE and the DTE's to assist in many instances. Additionally, BTE and the DTE's for a variety of reasons will desire to be "in" on certain arrangements on such contracts. These contacts are to be made through BTO. BTO alone shall give directions to all contractors.

12. Before any traffic signal (or flashing signal or other electrical device) is accepted and placed into operation, the project shall be inspected and approved by a team comprised of the staff of the Engineer, BTO, BTE, and the DTE.

13. Timing charts for all signals are to be the responsibility of the DTE's provided preliminarily with the Design Request and again in final form at the time of signal turn-on.

14. Other than for purposes of equipment maintenance, no one is to make any equipment alterations or functional or material design changes in any existing traffic signal. Signals may be checked out visually at any time and timing element adjustments may be made with reason, after acceptance for maintenance by BTO. (NOTE: It is stressed that only knowledgeable personnel should attempt to make any changes in the controller settings--and then only within those controllers that are intimately known. It is preferred that timing changes be given to the BTO staff to place in effect.)

15. All timing changes by anyone at any time shall be fully justified and supported by traffic engineering considerations, and shall be noted immediately on the timing card within the controller with
Chief, Bureau of Traffic Engineering  
Chief, Bureau of Traffic Operations  
District Traffic Engineers  
May 25, 1980  

Page 4

date, time, and name. (If there is no card, notify BTO immediately). A MEMO SHALL BE SENT IMMEDIATELY TO THE BTO INDICATING THE TIMING CHANGES, AND THE REASONS FOR THEM. A copy shall go to the District Engineer, the District Traffic Engineer, and any appropriate file.

16. A Preventative Maintenance program is to be developed whereby an annual call is to be made to each signal on the state highway system by BTO. An inspection and operation review is to be made similar to that made at the time of final inspection. A suitable record is to be kept by BTO.

17. If equipment failure is noted during any maintenance call, the district office is to be provided a copy of the maintenance report describing the nature of the call and the disposition. If a component cannot or will not be repaired, the district office is to be so advised.

18. An Annual Functional Operation Review will be made of each signal on the state highway system by the BTE's. A TM count and accident experience analysis will be made along with a review of the other appropriate traffic and highway characteristics. A review of the total area traffic operations will also be made, to the extent necessary that it is determined that a change in the functional operation of the signal or system is or is not called for. Local jurisdictions and the Bureau of Traffic Projects staff will be called upon for input where necessary. A report will be filed with copies to the local jurisdiction, if local roadways are involved, and to the ACE-T. If functional changes are felt to be justified, the report will be submitted as in the case of new signal studies, for forwarding on to the OEU, BTE, and/or BTO as the case may be.

19. In the case of signals along state highways maintained by local jurisdictions, the BTO is to develop a procedure for the annual maintenance review by State Highway Administration staff. Further, the ACE-T shall, at the earliest practicable time, direct to the local jurisdiction the State Highway Administration's requirements for such maintenance, necessary records, and notification to the State Highway Administration of maintenance performance.
20. In the case of signals along state highways maintained by local jurisdictions, the DTE’s are directed to meet with local officials to develop a routine procedure for the undertaking of functional operation reviews. The reviews should be jointly held and the reports filed and/or handled as in item #18.

21. Traffic signal equipment shall be procured by BTO using normal purchasing procedures. Any deviation from these procedures shall first have the approval of the ACE-T.

22. All contracts, other than those within the SHA program, shall be approved by the ACE-T and then handled by the BTO. Normal procedures are to be followed to the maximum degree practicable. Such projects shall not normally be under district control during construction. The BTO shall keep the DE, DTE, OEU, and ACE-T fully apprised of all activity.

23. At the earliest possible time, BTE shall inform BTO as to specific equipment needs. Generally, much of this information will be derived from the Traffic Signal Program.

24. A Traffic Signal Program shall be developed by the OEU listing all projects approved by SHA management in line with listings and priorities set by the Districts and concurred with by the ACE-T. This program will be the Traffic Division’s program and will be used by all Division units for their own programming and for informing others outside of the Division as to approximate dates for accomplishment. In this regard, the dates noted on the program are the official dates and none other shall be used. All dates are subject to significant revision due to fluctuating resources and the possibility of higher priorities occurring.

25. The Traffic Signal Program is to be reviewed and updated monthly.

26. BTO staff shall not change the functional operation of any signal without first checking with the DTE’s and/or BTE for necessary concurrence or direction.

27. BTO staff shall not normally change the timing of any signal. When such timing has been changed, it shall be done in compliance with the requirements of item #15.
28. Funding of traffic signal projects shall be as follows, unless otherwise directed by the ACE-T:

a. Within municipalities - 100% State - all charges.
b. With County Roads - % share by leg, energy by County, maintenance by State, regardless of newness of roadway.
c. Developers - 100% by others, energy by developer, maintenance by State.

Every unit in the Traffic Division is expected to fulfill its responsibilities with complete cooperation with other TD units. We must strive for accurate and complete work accomplished in a correct and efficient manner. Our program must be well disciplined and flexible to meet changing conditions. Mutual respect for each other’s responsibilities and skills will assure that each of our tasks supplements the other’s.

Please let me know of any comments or questions that you may have.

Thob Thomas Hicks

cc: Mr. C. E. Buck
    Mr. R. J. Bush
    Mr. G. E. Cook
    Mr. P. S. Jaworski
    Mr. W. K. Lee, III
    Mr. W. F. Lina, Jr.
    Mr. A. L. Gardner
    District Engineers
B. SHA Policy for Determining Yellow Timing at Intersections

MEMORANDUM

TO: Distribution List
FROM: Thomas Hicks, P.E., Director
Office of Traffic & Safety
DATE: March 20, 2003
SUBJECT: SHA Policy for Determining Yellow Timing at Intersections

Attached, please find a synopsis of the State Highway Administration’s (SHA) adopted policy for determining yellow timing at signalized intersections. An easy to read chart is included which shows the yellow duration according to the relative vehicle approach speed and grade of the roadway.

These calculations are directly from 1999 edition of the Institute of Transportation Engineer’s (ITE) Traffic Engineering Handbook. All fractional results for this calculation method have been rounded up to the nearest half second.

More detailed information or this topic including sample calculations and other state’s policies are presented in a report prepared by Bellomo McGee, Inc. for the Office of Traffic & Safety. Copies of this report can be obtained by calling Eric Tabacek.

If you have any questions or require additional information related to this policy, please do not hesitate to contact me or Eric Tabacek. He may be reached at 410-787-5860 or etabacek@sha.state.md.us.

Thank you.

TH/eth
Attachments

cc: Mr. Eric Tabacek
Maryland State Highway Administration’s
Policy for Determining Yellow Timing
at Signalized Intersection

Prepared by: BMI Consultants
for the Office of Traffic & Safety
March 20, 2003
Background

The function of the yellow change interval is to warn vehicle traffic that there is an impending change in the right-of-way and that a RED signal indication will be exhibited immediately thereafter (with the exception of left-turn signals operating in protected-permissive phasing). The yellow change interval should be of sufficient length to allow time for the motorist to see the YELLOW signal indication and decide whether to stop or to enter the intersection. It should allow motorists further away from the signal to decelerate comfortably in advance of entering the intersection and motorists closer to the signal to at least enter the intersection.

Equations and Recommended Values

The yellow change interval, \( Y \), should be calculated with the following equation:

\[
Y = t + \frac{V}{2(a + Gg)}
\]

[3]

Where

\[
\begin{align*}
Y &= \text{length of the yellow interval in seconds;} \\
 t &= \text{driver perception/reaction time, typically 1.0 second;} \\
 V &= \text{velocity of approaching vehicle in ft/s;} \\
 a &= \text{deceleration rate, typically 10 ft/s}^2; \\
 G &= \text{acceleration due to gravity, 32 ft/s}^2; \text{ and} \\
 g &= \text{grade of approach, in percent divided by 100 (downhill is negative).}
\end{align*}
\]

If available, the 85th percentile speed should be used as the approach speed in this equation. If the 85th percentile speed is not available, the engineer should use their judgment of the prevailing intersection approach speed. In no case should the approach speed used in the calculation be less than the posted speed limit.

If the approach has more than 15% heavy vehicles, the engineer should consider a slower deceleration rate to accommodate the reduced stopping ability of heavy vehicles. Based on the literature and what other agencies have adopted, 8 ft/sec\(^2\) is recommended. The engineer should consider the need to accommodate heavy vehicles versus the impact of the longer yellow interval on the capacity of the intersection.

The minimum yellow interval should be 3.5 seconds. For ease of implementation, the calculated yellow interval should be rounded up to the nearest half second. The maximum yellow interval should be 6.0 seconds. If the calculated yellow interval is longer than 6.0 seconds, a yellow interval of 6.0 seconds should be used and the remaining calculated time should be added to the all-red interval.

Table 1 presents some calculated values of the yellow interval for various approach speeds and grades. Table 2 presents some calculated values of the yellow interval when
there are more than 15% heavy vehicles on the approach and a slower deceleration rate (8 ft/sec²) is needed.

Other Considerations

The engineer should consider other aspects of the intersection operation besides those explicitly accounted for in the equation when timing the change period.

Driver Expectation

Drivers have some expectation of what the length of the yellow interval should be based on their past driving experiences. Specifically, along corridors or arterials with coordinated signal progression, the yellow interval should be constant. Within some reasonableness, the longest yellow interval needed in the corridor should be applied.

Implementation

The engineer should apply a uniform yellow interval for both approaches of a roadway to an intersection. For example, when one approach has a positive approach grade and the other approach has a negative approach grade, the more conservative (i.e., longer) yellow interval should be used for both approaches. These considerations will not only accommodate driver expectation, but also ease implementation of the signal timing. It will reduce the opportunity for error.

Left-Turn Phasing

When there is a separate left-turn phase at an intersection, the calculation of the yellow interval for the left-turn phase is the same as the calculation for the through vehicles. This is a more conservative yellow interval. If needed, the interval could be shorter. However, using the same yellow interval for the left-turn phase and the through movement will ease implementation and driver expectancy.

If a lagging left-turn phase is used concurrently with the associated through, the change interval needed for the left-turn phase and the change interval needed for the through phase should both be calculated. The greater of the two values should be used.
<table>
<thead>
<tr>
<th>Approach Speed (in MPH)</th>
<th>5% Uphill</th>
<th>Level</th>
<th>5% Downhill</th>
<th>10% Downhill</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
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<td>5.5</td>
<td>6.0 + 0.5*</td>
</tr>
<tr>
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<td>6.0 + 1.0*</td>
</tr>
<tr>
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<td>6.0 + 1.5*</td>
</tr>
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<td>6.0</td>
<td>6.0 + 1.0*</td>
<td>6.0 + 2.0*</td>
</tr>
</tbody>
</table>

*Use 6.0 seconds for the yellow interval and add the remainder, for example + 0.5, to the all-red interval. The maximum change interval (Yellow + All-Red) should not exceed 9 seconds.
### Table 2: Yellow Interval Durations for Approaches with Greater than 15% Heavy Vehicles for Various Approach Speeds and Grades

<table>
<thead>
<tr>
<th>Approach Speed (in MPH)</th>
<th>Yellow Interval Duration (in seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FOR APPROACHES WITH &gt;15% HEAVY VEHICLES</td>
</tr>
<tr>
<td></td>
<td>5% Uphill</td>
</tr>
<tr>
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<tr>
<td>30</td>
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</tr>
<tr>
<td>60</td>
<td>6.0</td>
</tr>
</tbody>
</table>

*Use 6.0 seconds for the yellow interval and add the remainder, for example + 0.5, to the all-red interval. The maximum change interval (Yellow + All-Red) should not exceed 9 seconds.
### DURATION OF YELLOW CHANGE INTERVAL

<table>
<thead>
<tr>
<th>Speed Limit</th>
<th>Approaches with Less Than 15 Percent Heavy Trucks</th>
<th>Approaches with 15 Percent or Greater Heavy Trucks</th>
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</table>

<table>
<thead>
<tr>
<th>Speed Limit</th>
<th>Acceleration in feet/sec²</th>
<th>Reaction Time in Seconds</th>
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<tr>
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### Percent Grade (Positive is Downgrade)

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<tbody>
<tr>
<td>Duration</td>
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</table>
Proposed COMAR Regulation for Establishing
The Minimum Length of Yellow Change Intervals at
Traffic Signals Where Traffic Signal Monitoring Systems
Are in Operation

DRAFT June 26, 2003

Authority: 2003 Laws, Chapter 218, to be codified as Transportation
Article, §21-202.1(b).

1. This regulation applies to each traffic signal within the state where a traffic signal
monitoring (red light camera) system is in operation.

2. The government agency or other entity responsible for the operation of the traffic
signal shall ensure that the yellow change interval on each approach where a
traffic signal monitoring system (red light camera) is in operation is no shorter in
duration than that shown in the following tables for the appropriate approach
speed and grade.

3. Table 1 shall be used where the percentage of heavy trucks on the approach is 15
percent or less of the total motor vehicles on the approach. Table 2 shall be used
where the percentage of heavy trucks on the approach is greater than 15 percent of
the total motor vehicles on the approach.

4. In using either of these tables, the 85th percentile speed, if available, shall be used
as the approach speed. If the 85th percentile speed is not available, engineering
judgment as to the prevailing intersection approach speed shall be used as the
approach speed.
Table 1: Minimum Yellow Change Interval Duration for Various Approach Speeds and for Various Approach Grades Where Heavy Trucks Comprise 15 Percent or Fewer of the Approaching Vehicles

<table>
<thead>
<tr>
<th>Approach Speed (in MPH)</th>
<th>5% Uphill</th>
<th>Level</th>
<th>5% Downhill</th>
<th>10% Downhill</th>
</tr>
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<td>6.0 + 1.0*</td>
<td>6.0 + 2.0*</td>
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</tbody>
</table>

*Use 6.0 seconds for the yellow change interval and add the remainder (shown as the + value) to the red clearance (all red) interval. The total duration of the combined yellow change and red clearance intervals should not exceed 9 seconds.
Table 2: Minimum Yellow Change Interval Duration for Various Approach Speeds and for Various Approach Grades Where Heavy Trucks Comprise Greater Than 15 Percent of the Approaching Vehicles

<table>
<thead>
<tr>
<th>Approach Speed (in MPH)</th>
<th>5% Uphill</th>
<th>Level</th>
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*Use 6.0 seconds for the yellow change interval and add the remainder (shown as the + value) to the red clearance (all red) interval. The total duration of the combined yellow change and red clearance intervals should not exceed 9 seconds.
C. SHA Part-Time Signal Operation Worksheet

CONDITON DIAGRAM

General Points to Consider

★ The guidelines/checklist should be met for each hour of proposed flashing operation.
★ Flashing Operation should be restricted to one continuous duration per day.
★ At a unique location the flashing operation should be restricted to no more than three separate periods per day. The preferred minimum period of flashing operation should be at least three consecutive hours.
★ Signal Operation should be changed to regular operation if accident pattern or severity increases or there is an increase in conflicts.
★ Sufficient fully operating signals should be maintained along highways in the urban areas to avoid a "Speedway" effect.
★ Criteria denoted with "★" must be satisfied in order to pursue flashing operation.
## PART-TIME FLASHING SIGNAL OPERATION

### Geometrics

<table>
<thead>
<tr>
<th>Question</th>
<th>Yes</th>
<th>No</th>
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<tbody>
<tr>
<td>Is the sight-distance adequate? (According to AASHTO standard)</td>
<td></td>
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</tr>
<tr>
<td>Is this a normal or simple intersection without extreme skews? (Not less than 75°)</td>
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<tr>
<td>Does the intersection have four or less approaches (legs)?</td>
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<tr>
<td>For undivided major roadway, is the intersection less than 90 feet wide?</td>
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<tr>
<td>Does the side road approach have more than one separate left turn lane?</td>
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### Volume

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<thead>
<tr>
<th>Question</th>
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<tbody>
<tr>
<td>Does the minor street approach carrying the higher volume have less than 50 VPH?</td>
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<tr>
<td>For a two lane major roadway, is the total two-way traffic volume less than 200 VPH?</td>
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<tr>
<td>For a multilane highway, is the total two-way traffic volume less than 300 VPH?</td>
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<tr>
<td>Is the ratio of major street to minor street volume greater than 4:1?</td>
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<tr>
<td>Does the minor street traffic stream consist mainly of passenger cars?</td>
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<tr>
<td>Is pedestrian activity at the intersection insignificant during the proposed flashing operation period?</td>
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### Accident

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<td>Has the intersection had 3 or less angle accidents during the last 12 months for the proposed flashing operation period?</td>
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<tr>
<td>Are the nighttime accidents at the intersection less than statewide average of the total accidents?</td>
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### Flashing Operation should be restricted to one continuous duration per day.

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<td>Does the intersection operate independently (not in system) during the proposed flashing operation period?</td>
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<tr>
<td>Does the intersection operate without pre-emption?</td>
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### Comments/Conclusions

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D. **SHA Signal Timing Chart**

![SHA Signal Timing Chart](image)
### Controller Submenu

#### 1. Phase Overlap Assignments

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### 1. NIC/TOD Submenu

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<th>LOCAL</th>
<th>LOCAL SYSTEM DET NUMBER</th>
</tr>
</thead>
<tbody>
<tr>
<td>DETECTOR NUMBER</td>
<td>1 2 3 4 5 6 7 8</td>
</tr>
<tr>
<td>NUMBER</td>
<td></td>
</tr>
<tr>
<td>NUMBER</td>
<td></td>
</tr>
</tbody>
</table>
E. SHA Capacity Procedures for Intersections – Critical Lane and Queuing Analysis

MEMORANDUM

TO: ALL Assistant District Engineers - Traffic

FROM: Thomas Hicks, Deputy Chief, Engineer
Office of Traffic

SUBJECT: Capacity Procedures for Intersections
Critical lane Analysis
Queuing Analysis

Enclosed are procedures/Methcdology to be used in performing critical lane analysis at intersections. These procedures were developed to ensure some continuity between the agencies throughout the state.

If there is any questions concerning these procedures, please contact William Richardson at 787-5665.

TH/mze

cc: All District Engineers
Mr. Neil Pederson
Mr. Joe Finkle

ATTACHMENT B

My telephone number is (301) _______________
CAPACITY PROCEDURES INTERSECTIONS
CRITICAL LANE ANALYSIS

INTRODUCTION

This section describes the procedures for the capacity analysis of intersections. They include some revision but rely heavily on early work in this area by McInerney and Petersen, and later efforts by JHK and Associates and material contained in the 1985 HCM. The methodology described will fit most intersection configurations and can be varied easily for special situations and unusual conditions. This method applies at isolated intersections or any other location where the operation is not radically affected by adjacent traffic signals. Modifications to this procedure or use of methods suggested for arterials are required in those instances.

DATA REQUIRED

The following data is necessary to ensure reasonable results:
- intersection turning movement counts or projections
- intersection geometrics
- lane assignments
- special operating characteristics

Additional information may be needed at some sites such as:
- pedestrian timing
- typical queue length
- lengths of turn lanes
- offsets or clearance times from adjacent signals

The procedures are based on typically occurring situations subject to modification based on the data described above. Further adjustments are possible based on observed impacts of factors such as frequent bus stops, very large truck volumes, bikeway influences and so forth.
CAPACITY PROCEDURES INTERSECTIONS

PROCEDURES

General Case

(1) Enter the available data on the worksheet.

(2) Utilizing the appropriate Lane Use Factors determine the critical lane volume which controls the green time required for each signal phase.

(a) For a two-phase signal operation the opposing lefts are added to the factored thru/right lane volume. The assumption is made that although they move on the same phase as thru traffic these left turns required green time within the phase. Two-phase signals will be assumed except where existing phasing is different or double left/right turn lanes are being provided. An additional phase or phases must be included in the latter cases.

(b) Each major movement occurring during the signal phase is calculated separately with the larger sum being noted as critical (*) in controlling the time required for the phase.

(c) Where left turn phasing includes no overlaps, the left turn and thru phases must be calculated separately.

(d) Where left turn phasing includes overlaps with the thru movements the procedures will be the same as described above for each phase.

(3) Sum the Critical Lane Volumes (*) for the assumed phasing to determine the Level-of-Service.

SPECIAL CASES

EXCLUSIVE RIGHT TURN LANES

Where the right lane is devoted to the exclusive use of right turn vehicles, a maximum lane volume should be computed separately for through movements and right turn movements. If a right turn phase overlap is provided with a left turn phase on the cross street, subtract the overlapping left turn volume from the right turn volume. The highest of the through or right turn lane volumes should be added to the opposing left turn volume, except where significant right turns on red occur.
CAPACITY PROCEDURES INTERSECTIONS

FREE RIGHT TURNS

A free right turn is one which is not controlled by the traffic signal or stop sign. Normally the movement is isolated by a channelizing island and controlled by a yield sign. If the right turn movement is serviced by an exclusive right turn lane of sufficient length that right turning vehicles are not part of the queue of thru vehicles, the right turning volumes can be excluded from the critical lane analysis. Knowledge of the intersection can be used to combine a sufficient number (or percent) of the right turns with the thru traffic to reflect actual peak hour operations. In the absence of such knowledge a queuing analysis could be done. As a rule-of-thumb 150 feet of exclusive right turn lane will permit excluding all right turns; less than 50 feet will require that all rights be included. Distances within that range suggest that a portion of the right turn volume be included.

RIGHT TURN ON RED

The number of vehicles which can take advantage of the RTOR feature vary greatly based on site and traffic characteristics. At higher volume intersections, as the Level-of-Service diminishes, few gaps are generally available for RTOR. Unless observations of the RTOR operations support excluding some right turns from the Critical Lane Analysis, this feature will normally not be considered.

NO SEPARATE LEFT TURN LANE

On multi-lane approaches with no separate left turn lane the impact of left turning traffic may be significant, especially on high volume roadways. Typically the left lane operates as a left turn lane with nearly all thru traffic avoiding this lane. Calculations for such an approach should be as follows:

The left turn volume will be adjusted using the PCE Factor of the 1985 HCM Pages 9-35. The opposing volume will be total thru’s and rights.

When the adjusted left turn volume is greater than the remaining volume being included in the analysis, the left most lane will be considered an exclusive left turn lane. The analysis will proceed with that assumption. For other cases the resulting left turn volume will be added to the rest of the approach volume and the appropriate lane use factor applied to the total.
CAPACITY PROCEDURES INTERSECTIONS

ON-LANE APPROACHES

Where a bypass of left turning vehicle is available the one lane approach should be treated as if there is a separate left turn lane. If no bypass area is available traffic on the one lane approach can proceed only when there is no vehicle waiting to turn left. This case should be analyzed using PCE equivalencies (1985 HCM pages 9-35) to modify the left turn volumes. The resulting total will be added to the rest of the approach volume and the appropriate lane use factor applied.

DOUBLE LEFT TURN LANES

Both the access to the double left turn lane and movements made immediately after the left turn will influence the distribution of traffic between the available lanes. Generally the distribution is less balanced than for thru lanes; thus the recommended lane use factor of 0.60. Variations observed at specific sites may suggest the use of different factor for this movement.
QUEUING ANALYSIS

Procedures for determining queue lengths at signalized and unsignalized intersections:

Signalized Intersections

This procedure can be used at intersections with existing signals and intersections where it is felt a signal may be installed.

- Perform critical lane analysis
- Select cycle length
  - use existing timing if available
  - if timing is not available, use the suggested cycle lengths

Recommended Maximum Cycle Lengths

<table>
<thead>
<tr>
<th>LOS</th>
<th>2 Phase</th>
<th>3-5 Phase</th>
<th>6-8 Phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>90</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>B</td>
<td>90</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>C</td>
<td>100</td>
<td>120</td>
<td>135</td>
</tr>
<tr>
<td>D</td>
<td>120</td>
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</tr>
<tr>
<td>E</td>
<td>135</td>
<td>150</td>
<td>165</td>
</tr>
<tr>
<td>F</td>
<td>150</td>
<td>165</td>
<td>180</td>
</tr>
</tbody>
</table>

Note: These cycle lengths are to be used as a guide, knowledge of the intersection may result in using a higher or lower cycle.

- Use Poisson Distribution Chart/Formula to determine maximum number of vehicles per cycle of a specific movement.
  
  formula: \( \frac{\text{Avg. Veh/Cycle}}{\text{Critical Lane Volume (veh/hr) x Cycle Length (Sec.)}} = \frac{1}{3600} \) (sec/hr)

- Assume a vehicle length of 25 ft.
- Once the average vehicles per cycle (specific movement) is determined, the chart can be used to find the maximum vehicles per cycle for that movement.
The queue length will be the maximum vehicles per cycle times 25 ft. per vehicle.

It is noted that the chart ends at an average of 20 vehicles per cycle. In cases where the average number of vehicles per cycle exceeds 20 the following formula can be used to determine the queue length. This formula can also be used in lieu of the chart.

\[ Q = \text{Avg. No. of Veh} \times 1.4 \times 25 \]
Unsignalized Intersections

This procedure can be used at isolated intersections where it is felt a signal will not be placed. If there is any chance that a signal may be placed at an intersection, the procedure for signalized intersections should be used.

Determine the critical gap needed for the movement (from chart) this chart is also found in the 1985 HCM unsignalized intersections.

**BASIC CRITICAL GAP FOR PASSENGER CARS, SEC**

<table>
<thead>
<tr>
<th>Vehicle Maneuver and Type of Control</th>
<th>Average Running Speed, Major Road</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>30 MPH</td>
</tr>
<tr>
<td>RT from Minor Road</td>
<td>2</td>
</tr>
<tr>
<td>STOP</td>
<td>5.5</td>
</tr>
<tr>
<td>YIELD</td>
<td>5.0</td>
</tr>
<tr>
<td>LT from Major Road</td>
<td>5.0</td>
</tr>
<tr>
<td>Cross Major Road</td>
<td>2</td>
</tr>
<tr>
<td>STOP</td>
<td>6.0</td>
</tr>
<tr>
<td>YIELD</td>
<td>5.5</td>
</tr>
<tr>
<td>LT from Minor Road</td>
<td>2</td>
</tr>
<tr>
<td>STOP</td>
<td>6.5</td>
</tr>
<tr>
<td>YIELD</td>
<td>6.0</td>
</tr>
</tbody>
</table>

**Note:** If restricted sight distance exists add one second to the gap needed. Where average running speeds are between 30 mph and 55 mph, interpolate.

- Determine average gap between opposing vehicles

\[
\text{Average Gap Opposing Vehicle} = \frac{3600 \text{ sec}}{\text{Volume Per Hour}}
\]

- If the average gap is greater than the gap needed for the maneuver the same procedure as signalized intersections can be used with the cycle length equal to the critical gap required (from chart) plus 4 seconds. (start up time).

- If the average gap is less than or equal to the gap needed, this maneuver should be analyzed as if a signal were in place.
### F. Isolated Intersection Signal Timing Checklist

#### ISOLATED INTERSECTION SIGNAL TIMING CHECKLIST

<table>
<thead>
<tr>
<th>Main Road</th>
<th>Side Road</th>
<th>County</th>
<th>District</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Observed By**

<table>
<thead>
<tr>
<th>Speed Limit (Major)</th>
<th>Speed Limit (Side)</th>
<th>Date</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. Do queues clear each cycle?  **Yes** / **No**  if no, specify ______________________________
2. Do phases “max out” or “gap out” each cycle?  ______________________________
3. Do left-turn or right-turn queues extend beyond storage?  **Yes** / **No**  if yes, specify ______________________________
4. Do through queues block the turn bays?  **Yes** / **No**  if yes, specify ______________________________
5. Are detection devices functioning properly?  **Yes** / **No**  if no, specify ______________________________
6. Are phases recalled even if demand is not present?  If so, are there “false calls” or are the recalls programmed?  **Yes** / **No**  if yes, specify ______________________________
7. Are signals functioning properly?  **Yes** / **No**  if no, specify ______________________________
8. What is the main road and side road signal phasing (e.g., permissive only, exclusive/permissive, exclusive, split)?  ______________________________
9. Does the exclusive portion of the exclusive/permissive phase(s) need additional green time?  **Yes** / **No**  if yes, specify ______________________________
10. Are signal heads aligned properly?  **Yes** / **No**  if no, specify ______________________________
11. Are all pedestrian timings adequate and are the pedestrian indications functioning properly?  **Yes** / **No**  if no, specify ______________________________
12. What are the field-measured crosswalk distances?  ______________________________
13. Is the push-button device easily accessible and functional?  **Yes** / **No**  if no, specify ______________________________
14. Are the APS features functioning properly?  **Yes** / **No**  if no, specify ______________________________
15. Is the timing chart available?  **Yes** / **No**  if no, specify ______________________________
16. Do the controller settings match the timing chart?  **Yes** / **No**  if no, specify ______________________________
17. Are all pavement markings adequate?  **Yes** / **No**  if no, specify ______________________________
18. Are overhead street name signings adequate?  **Yes** / **No**  if no, specify ______________________________
19. Are there route markers and advance warning signs on the approaches?  **Yes** / **No**  if no, specify ______________________________
20. Is the intersection lighting functioning properly?  **Yes** / **No**  if no, specify ______________________________
21. Should part-time flashing signal operation be adjusted?  **Yes** / **No**  if yes, specify ______________________________
22. General operation of the signal:
    Are there any perceived safety problems/conflicts?  ______________________________
    Are nearby access points/driveways impacted by traffic at the signal?  ______________________________

**Comments:**  ______________________________
GLOSSARY

**Actuated control (fully-actuated control)** – A type of signal operation in which all of the signal phases function on the basis of detection.

**Added initial** – A volume-density interval that times concurrently with the minimum green interval and increases by each vehicle actuation received during the associated yellow and red intervals. This time cannot exceed the maximum initial.

**Advance preemption** – The notification of approaching rail traffic is forwarded to the highway traffic signal controller prior to the activation of the railroad or light rail transit warning devices.

**Advance warning flasher** – A specific type of active hazard identification beacon that is installed upstream of a conventional traffic signal to warn approaching motorists prior to the onset of the yellow change interval. The assemblies typically include a flashing RED SIGNAL AHEAD sign or a SIGNAL AHEAD – PREPARE TO STOP WHEN FLASHING sign with yellow warning beacons.

**Advanced pedestrian phase (leading pedestrian phase)** – A type of pedestrian walk interval that begins several seconds prior to the vehicular phase for the adjacent vehicular movements to allow pedestrians to establish a presence in the crosswalk and to reduce conflicts with concurrent turning vehicles.

**Alternate sequence** – A time-of-day signal controller parameter that modifies the signal sequence by reversing the order in which pairs of phases, e.g., phases 1 and 2 or phases 3 and 4, are serviced.

**Alternate vehicle extension (alternate passage time)** – A time-of-day signal controller parameter that enables the use of the auxiliary vehicle extension interval to increase or decrease the preset gap during a specific time period, such as school arrival and dismissal periods.

**Barrier** – The separation of intersecting movements to prevent operating conflicting phases simultaneously.

**Call** – An indication within a signal controller that a vehicle or pedestrian is awaiting service from a particular phase or that a recall has been placed on the phase.

**Capacity** – The maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions.

**Concurrent phasing (simultaneous phasing)** – Two or more phases in separate rings that operate together without conflicting movements.

**Conditional phase (lead plus lag phase or phase re-servicing)** – Servicing a left-turn phase twice per cycle, first as a lead phase and then as a lag phase, when the opposing through phase terminates early and the through phase in the same direction is still being serviced.

**Coordinated-actuated** – A type of signal operation that allows a coordinated phase to operate with a preset minimum split that can be extended each cycle using passage detection and the corresponding vehicle extension interval.

**Critical lane volume** – A simplified capacity analysis methodology that can be used to determine basic signal timing parameters, such as the maximum green interval, split, and cycle length.

**Cycle length** – The time required for one complete sequence of signal indications.
Delay

**Travel time delay**: The additional travel time experienced by a driver, passenger, or pedestrian.

**Detector delay**: A detector parameter, typically used with presence detection for turning movements from exclusive lanes, used to temporarily postpone a call for a phase.

**Demand** – The number of users desiring service on the highway system.

**Detector cross switching** – A setting that allows a detector to be assigned to different phases each cycle by initially assigning the detector to a phase then changing the phase assignment later in the signal sequence.

**Detector phase assignment** – A signal controller setting that allows each vehicle detector to be designated to any or all of the twelve phases and place a call on the designated phase(s).

**Detector type** – A signal controller setting that classifies each detector, such as normal/standard, extend/delay, stop bar, calling, bike, dilemma zone, or walk.

**Early green** – A term used to describe servicing a coordinated phase prior to its programmed offset as a result of unused time from non-coordinated phases.

**Exclusive phase (protected phase)** – A mode in which left or right turns are permitted when a left or right green arrow signal indication is displayed.

**Exclusive/permissive phase (protected/permitted phase)** – A combined left-turn treatment that displays the exclusive phase before the permissive phase.

**Exit maximum times** – The maximum green intervals in effect for one cycle following a preemption sequence for all phases except the hold phases. This parameter is intended to clear queues at the end of a preemption routine.

**Exit phases** – The phases serviced at the end of a preemption sequence that serve as transition phases to return to standard signal operations.

**Gap out** – When an actuated phase terminates prior to its maximum green time due to a lack of calls within the vehicle extension time.

**Gap reduction** – A volume-density feature that reduces the vehicle extension time to a smaller value while the phase is active.

**Greenband (bandwidth)** – The maximum amount of green time for a designated direction as it passes through a coordinated signal system at an assumed constant speed.

**Half cycle length (double cycling)** – A cycle length that allows phases to be serviced twice as often as other intersections in the coordinated signal system.

**Hold phases** – The phases that remain in service during preemption until the hold time expires or until the preemption call is dropped.

**Inductive loop** – An intrusive type of vehicle detection that can be used for presence and passage detection. The detectors are formed by sawcutting the roadway, placing a #14 AWG wire incased in flexible tubing into the sawcut, and then sealing the sawcut.

**Lag phase** – An exclusive left-turn phase that operates concurrently with the through phase in the same direction and that is sequenced after the opposing through phase.
**Lead phase** – An exclusive left-turn phase that operates concurrently with the through phase in the same direction and that is sequenced before the opposing through phase.

**Lead/lag phasing** – A left-turn treatment where one left-turn movement begins with the adjacent through phase and then the opposing left-turn movement begins at the end of the opposing through phase.

**Leading flash time (AWF overlap interval)** – The time prior to the onset of the yellow change interval that the advance warning flasher is activated to warn approaching motorists of the upcoming phase termination.

**Locking memory** – A controller mode used to place a call for service after the first actuation is received by the controller during the red interval. The controller retains the call even if the vehicle departs the detection zone.

**Master-slave system (relay based system)** – A signal system that consists of two intersections hardwired together with an interconnect cable, where one intersection operates fully-actuated and corresponding holds/controls phases at the adjacent intersection.

**Max extension** – A signal controller parameter that will automatically increase the maximum green interval if a phase terminates due to “max out” during two successive cycles.

**Max out** – When an actuated phase terminates at its preset maximum green time.

**Maximum green interval** – The maximum time a phase can be green in the presence of a conflicting call.

**Microloop probe** – A small, cylindrical, passive transducer that is placed underneath the roadway in a nonmetallic conduit and that detects vehicles by sensing disruptions in the earth’s magnetic field.

**Minimum gap** – A volume-density parameter that specifies the minimum vehicle extension when gap reduction is used.

**Minimum green interval** – The first timed portion of the green interval which may be set in consideration of driver expectancy or based on the storage of vehicles between the detectors and the stop line.

**Model 170 controller** – A type of signal controller that contains a user-programmable microprocessor instead of a standard NEMA ring-and-barrier structure.

**NEMA phasing (National Electrical Manufacturers Association phasing)** – A common ring-and-barrier controller structure that organizes phases by prohibiting conflicting movements from timing concurrently while allowing non-conflicting movements to operate simultaneously.

**Non-interconnected system (time-based coordinated)** – A basic type of signal system consisting of two or more signalized intersections coordinated solely by time.

**Non-locking memory** – A controller mode that retains a call for service only while the vehicle remains within the detection zone.

**Offset** – The time relationship between coordinated phases’ defined reference point (typically the beginning of green) and a defined master reference.

**On-street master** – An optional component of a coordinated signal system that facilitates coordination with the use of a local controller instead of a remote, central-based computer.
Opticom™ – A common type of preemption system that uses a visible light and/or infrared vehicle emitter and a receiver located adjacent to the traffic signal heads to identify and detect emergency vehicles.

Overlap phase – A green indication allowing two or more non-conflicting traffic phases to operate simultaneously.

Passage detector (pulse detector) – A type of detection where vehicle actuations are represented by a single “on” pulse to the controller.

Peak hour factor – A measure of the fluctuation in the demand within the peak hour.

Pedestrian clearance interval – An interval during which the flashing upraised hand (symbolizing don’t walk) pedestrian signal indication is displayed.

Pedestrian walk interval – An interval during which the walking person (symbolizing walk) pedestrian signal indication is displayed.

Permissive phase (permitted phase) – A mode in which left or right turns are permitted after yielding to pedestrians and/or opposing traffic.

Phase – A signal controller timing unit associated with the control of one or more movements.

Phase omit – A command, typically used during preemption or alternate sequences, which causes the exclusion of a selected phase.

Phase sequence – The order of a series of phases.

Preemption – The transfer of normal signal operations to a special control mode.

Preemption delay – The time between the receipt of the preemption call and the initialization of the preemption phases/sequence.

Presence detector – A type of detection where vehicle actuations are sent to the controller only while a vehicle is within the detection zone.

Pre-timed control – A type of signal operation in which the phase sequence, green times, and cycle length are preset to repeat continuously.

Progression (coordination) – The ability to synchronize multiple signalized intersections to enhance the operation of one or more movements in a signal system.

Queue – A line of vehicles, bicycles, or persons waiting to be served by the system.

Queue detector – A type of detector that is typically placed along freeway off-ramps upstream of signalized intersections to detect the presence of queued vehicles and to place a call on the signal controller to modify signal timings to prevent queues from extending back onto the freeway.

Recall – A repetitive call that is placed for a specified phase each time the controller is servicing a conflicting phase to ensure that the specified phase will be serviced each cycle. Types of recall include soft, minimum, maximum, and pedestrian.

Red clearance interval – An interval that follows the yellow change interval and precedes the next conflicting green interval.

Ring – A series of conflicting phases that operate sequentially.

Running time – The portion of the travel time during which the vehicle is in motion.
**Sampling detector (system detector)** – A type of detection that is placed downstream of signalized intersections to collect free flow traffic data, such as volume, speed, and occupancy.

**Saturation flow rate** – The equivalent hourly rate, in vehicles per hour per lane, at which vehicles can depart an intersection under prevailing conditions, assuming a constant green indication at all time and no lost time.

**Semi-actuated control** – A type of signal operation in which at least one, but not all, signal phases function on the basis of detection.

**Simultaneous preemption** – The notification of approaching rail traffic is forwarded to the highway traffic signal controller and railroad or light rail transit active warning devices at the same time.

**Split** – The time, expressed in seconds or percentage of the cycle length, assigned to a phase, typically the green plus yellow and all-red intervals, during coordinated signal operations.

**Split demand** – A signal controller function that can automatically change signal patterns to favor specific phases when excessive queues are detected along the approach.

**Split phasing (separate phasing)** – A type of exclusive phasing where opposing approaches operate separately, rather than simultaneously. Adjacent turning and through movements on the same approach operate simultaneously with this type of signal phasing.

**Telemetry system** – A signal system consisting of two or more intersections that are interconnected by a two-way communication system, and signal coordination is controlled by either an on-street master controller or a central-based computer.

**Time before reduction** – A volume-density timing period that begins when the phase is green and when there is a serviceable call on a conflicting phase. When this period expires, the linear reduction of the vehicle extension begins.

**Time-space diagram** – A chart that plots the location of signalized intersections along the vertical axis and signal timing intervals along the horizontal axis to serves as a visual tool for the coordination relationships among intersections.

**Time to reduce** – A volume-density timing period that begins when the time before reduction ends and that controls the linear rate of reduction of the vehicle extension until the minimum gap is achieved.

**Traffic adaptive** – A signal system control method where vehicles are detected at points upstream and/or downstream of signalized intersections and an algorithm is used to make automatic real-time signal timing adjustments.

**Traffic responsive** – A signal system control method which uses data from traffic detectors, rather than time-of-day information, to automatically select a preset timing plan best suited to current traffic conditions.

**Travel time** – The total elapsed time spent traversing a specified distance.

**V/C ratio (degree of saturation)** – The ratio of volume to capacity for a subject movement.

**Vehicle extension (passage time)** – A phase timer that terminates an actuated phase when the time between detector actuations exceeds the timer setting.

**Video detection** – A type of detection that uses video imaging to detect vehicles within a
camera’s field of view and predetermined detection zone.

**Volume** – The number of persons or vehicles passing a point on a lane, roadway, or other travel way during a specific time interval.

**Volume-density** – A type of actuated signal operation that incrementally increases the minimum green time based on actual traffic demand and that linearly reduces the vehicle extension time as the phase continues to time.

**Yellow change interval** – The first interval following the green or flashing arrow interval during which the steady yellow signal indication is displayed.
REFERENCES


Traffic Control Device Application Guidelines Manual, Maryland State Highway Administration.


