ACTIVITY 30

Considering Minimum Green Time, Passage Time, and Detection Zone Length

READING

Purpose

The purpose of this activity is to provide you with the opportunity to learn more about how the selection of the basic actuated traffic control timing parameters (minimum green time and passage time) are related to the length of the detection zone.

Learning Objective

- Describe the interaction of the minimum green time, the passage time, and the detection zone length in producing efficient intersection operation

Deliverables

- Define the terms and variables in the Glossary
- Prepare a document that includes answers to the Critical Thinking Questions

Glossary

Provide a definition for each of the following terms. Paraphrasing a formal definition (as provided by your text, instructor, or another resource) demonstrates that you understand the meaning of the term or phrase.

call	
detection zone	
interval	
maximum allowable headway	
occupancy time	

recall	
unoccupancy time	
h	
L_d	
L_v	
t_o	
t_u	
v	

Critical Thinking Questions

When you have completed the reading, prepare answers to the following questions.

1. Describe how passage time and the length of the detection zone are related.

2. What is one criterion for terminating a phase?

3. When using a standard loop detector with stop bar presence detection, why is it difficult to determine when a queue has cleared?

4. Explain why the passage time should be decreased when the detection zone length is increased.

5. Explain how variability in the vehicle lengths and speeds affect the determination of the passage time.

6. Describe in your own words the implications of the data presented in Figure 111.

7. Since vehicle headways vary widely and are not constant, even during periods of saturation flow, explain the risks involved in setting the passage time.

8. Summarize your understanding of the headway variability for the four time segments of vehicles departing after the start of green.

9. Describe how the problem of determining passage time changes when considering a two-lane approach.

INFORMATION

Introduction

Consider an actuated traffic control system with stop bar presence detection on a single-lane approach. A vehicle requests service by passing into the detection zone. The request is processed when the associated phase is next in the controller sequence. The timing of the phase is based on the interaction of the vehicle request (or call), the length of the detection zone, and the value of three timing parameters (the minimum green time, the passage time, and the maximum green time) as shown in the traffic control process diagram in Figure 107. In this example, six vehicles are stopped at the intersection at the beginning of green. They travel through the intersection when the green indication begins, as shown by the time-space trajectories, and the detection system responds to these vehicles (box at top left). The detector calls activate the controller timing processes (box at top right), and the signal displays respond to the controller timing processes and logic (box at bottom right). Finally, to "close the loop", the vehicles respond to the display status (box at lower left).

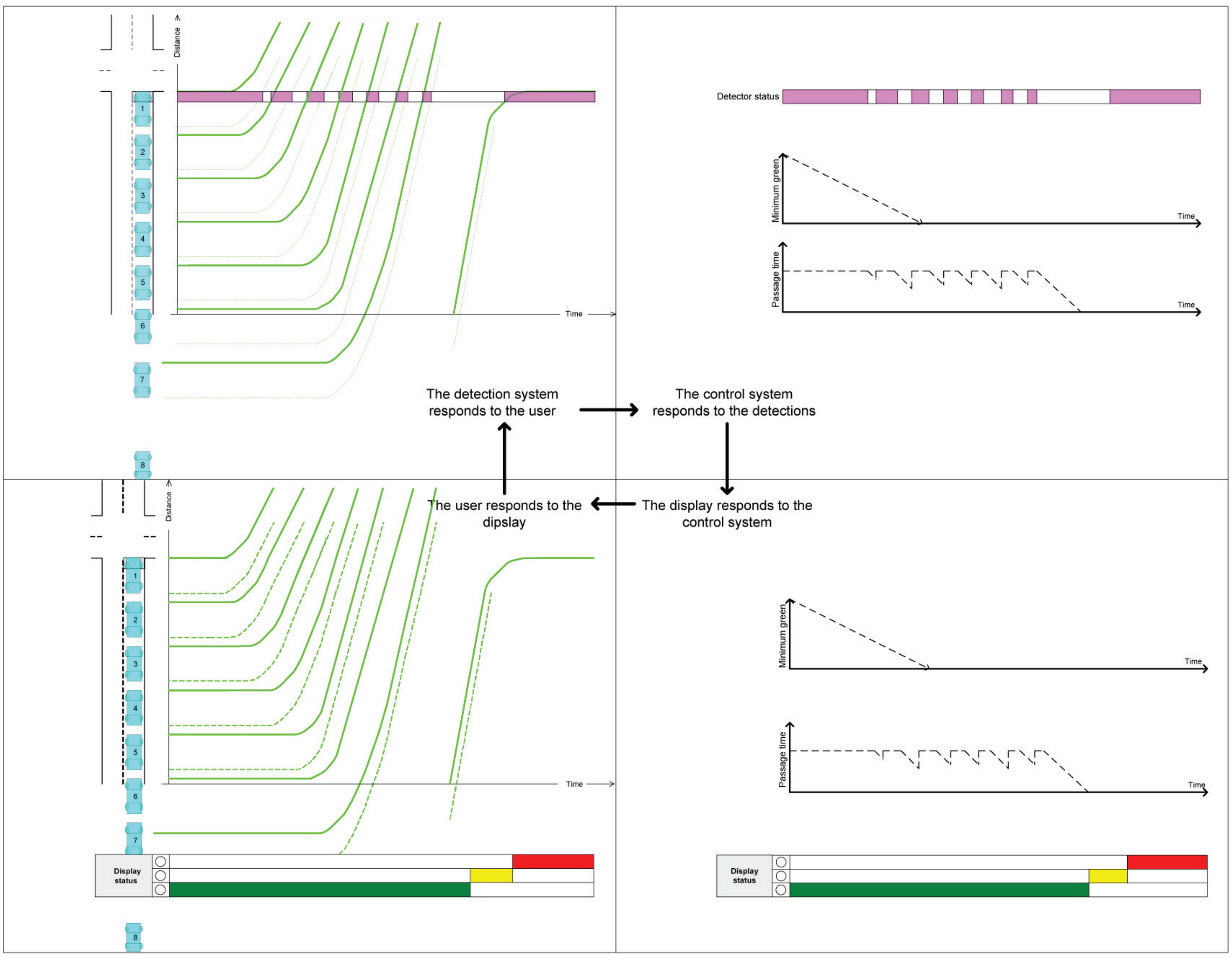


Figure 107. Traffic control process diagram

The purpose of the minimum green timing interval in the case of stop bar presence detection is to make sure the green is displayed for at least as long as driver expectation. Driver expectation can be thought of as a time so short that if the minimum green time is less than this time the public may complain or identify what they perceive to be a problem. The minimum green time does not have to account for a slow moving vehicle because as long as the vehicle is in the detection zone the call will be extended until it clears the stop bar. The minimum green time is generally lowest for left turns, more for side streets, and longest for through movements on the main street.

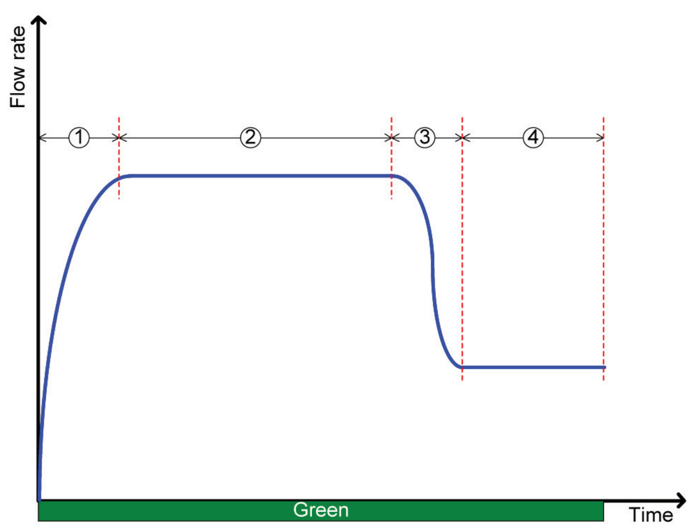
The purpose of the passage timer is to extend the phase as long as the headway between vehicles is less than a specified value called the maximum allowable headway (MAH). The goal is to make sure that green is displayed as long as a queue is present but then to terminate the phase when the queue has cleared.

In this chapter you will learn more about two of these timing parameters, the minimum green time and the passage time. Activities related to the maximum green time are included in Chapter 7.

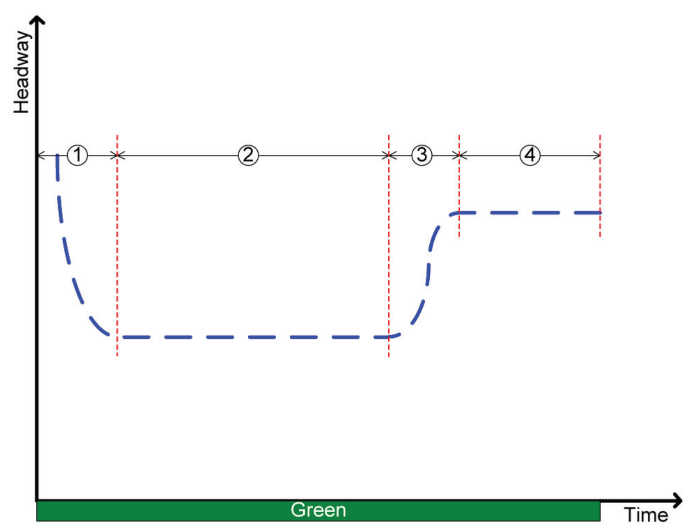
A Theoretical Foundation: Traffic Flow Theory for Queue Clearance

The traffic flow process of queue clearance can be represented as a flow profile diagram, as shown in Figure 108a. In the first segment of the green interval (noted as "1" in the figure), the flow rate increases as the queue begins to move from the stop bar into the intersection. This time period is characterized by the start-up lost time parameter described in the Highway Capacity Manual (HCM), and includes the first four vehicles in queue. In the second segment of the green interval, beginning with the fifth vehicle in queue, vehicles depart at the saturation flow rate. The HCM suggests an ideal value of 1900 vehicles per hour of green for the saturation flow rate.

When the queue has cleared, vehicles arrive at and depart from the intersection at a constant rate, with no delay. This is shown as the fourth segment of the green interval where the flow is represented as uniform. The segment between the queue clearance period (the second segment) and the period after the queue has cleared (the fourth segment) is the transition, or third, segment. It is during this third segment, after the queue has been served, that the phase should be terminated and service transferred to the next phase in the controller sequence. This process can also be represented in terms of headways, as shown in Figure 108b. The headway during the second segment is the saturation headway.



a. Departure flow profile diagram

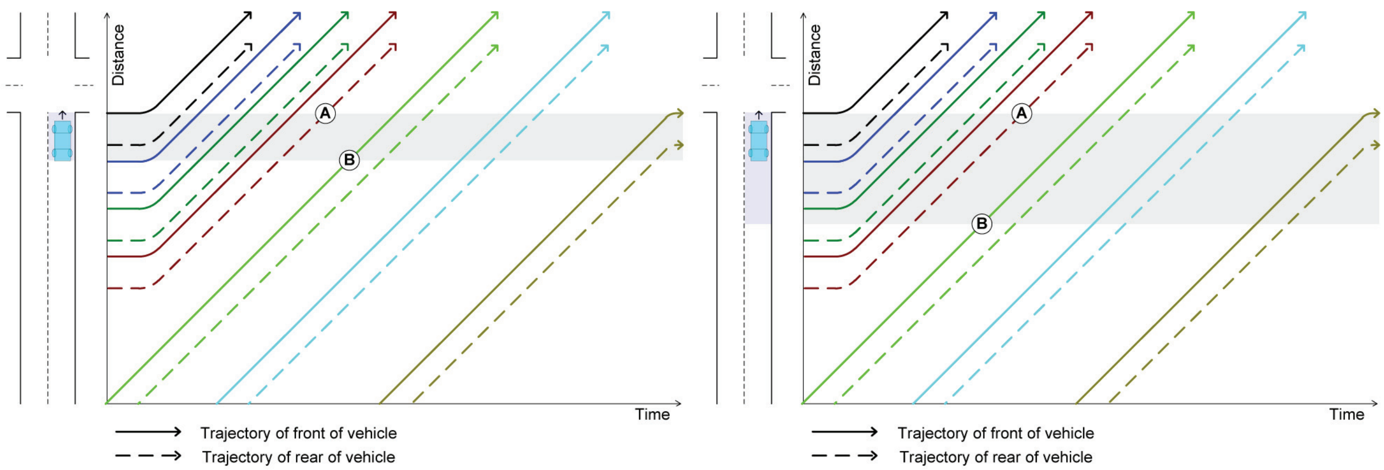


b. Departure headway profile diagram

Figure 108. Departure flow and headway profiles

Headway and Unoccupancy Time

In the field, traffic control systems don't typically measure flow rates or headways but rather whether the detection zone is occupied or not. Unoccupancy time is defined as the time that the detection zone is not occupied by a vehicle. Figure 109 shows a time space diagram representation of vehicles departing in a queue at the beginning of green and then arriving and departing without delay after the queue has cleared for two different detection zone lengths (shown in gray shade). When the unoccupancy time reaches the passage time set in the controller, the phase is terminated. The unoccupancy time depends directly on the length of the detection zone, as well as the vehicle speed (which may vary over time) and vehicle length. In Figure 109a, with a shorter detection zone, the horizontal distance between points A and B represents the unoccupancy time, the time between the fourth vehicle leaving the detection zone and the fifth vehicle arriving in the detection zone. In Figure 109b, with a longer detection zone, the event represented by point B occurs before that represented by point A (vehicle 5 arrives in the detection zone before vehicle 4 leaves the zone), so the unoccupancy time is zero.



a. Short detection zone **b.** Long detection zone

Figure 109. Time-space diagram for long and short detection zones

Maximum Allowable Headway (MAH)

Bonneson and McCoy (2005) established the concept of the MAH as the maximum headway that will be tolerated in the traffic stream before the phase is terminated. The analytical relationship between the unoccupancy time and the MAH can be developed as follows.

The headway (h) between two vehicles traveling on an intersection approach and through a detection zone consists of two parts, the time that the detection zone is occupied by the first vehicle and the time that the zone is unoccupied after the first vehicle leaves the zone and before the second vehicle arrives into the zone.

$$h = t_o + t_u$$

where t_o is the occupancy time and t_u is the unoccupancy time. The time that the detector is occupied (t_o) is equal to the length of the detection zone (L_d) plus the length of the vehicle (L_v), divided by the speed at which the vehicle is traveling (v).

$$t_o = \frac{L_d + L_v}{v}$$ Thus, we can write the unoccupancy time (t_u) as follows: $$t_u = h - \frac{L_d + L_v}{v}$$

This relationship is shown graphically in Figure 110. The occupancy time (t_o) is the time that it takes vehicle 1 to travel its own length plus the length of the detection zone. The unoccupancy time (t_u) is the time from when vehicle 1 leaves the detection zone until vehicle 2 arrives in the detection zone. The headway (*h*) is the sum of the occupancy time and the unoccupancy time.

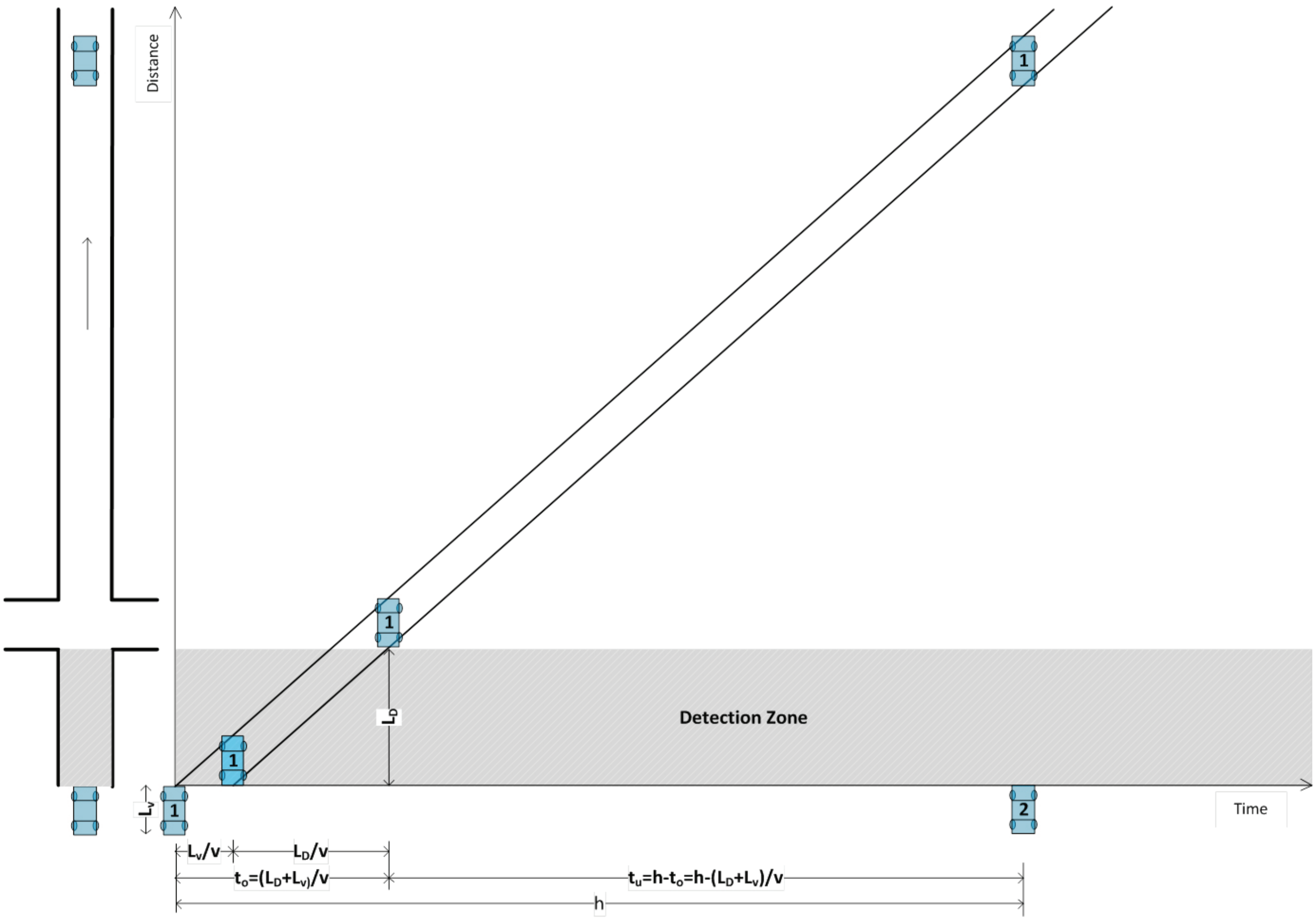


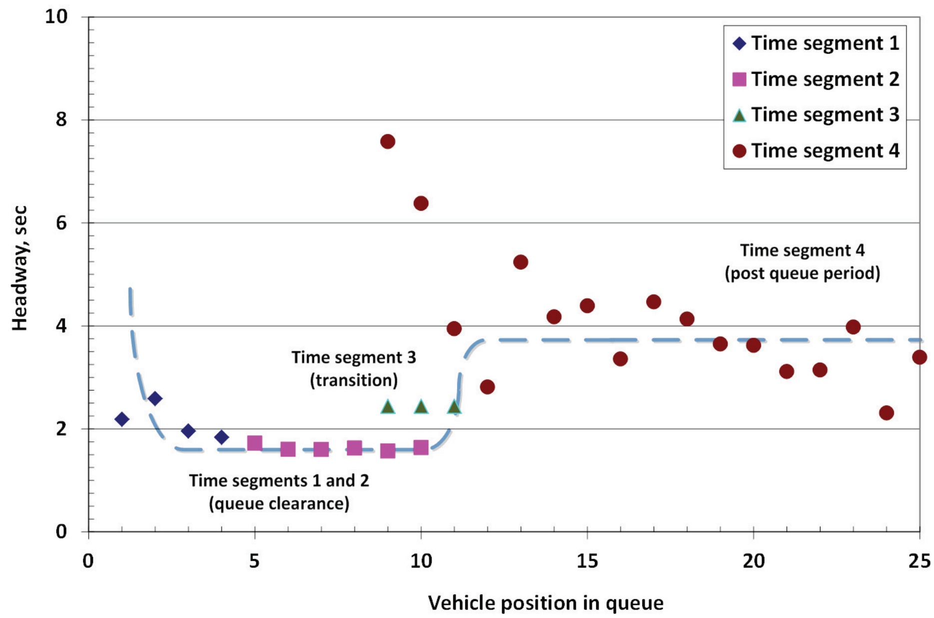
Figure 110. Headway, occupancy time, and unoccupancy time

A Stochastic Perspective Using Simulation Data

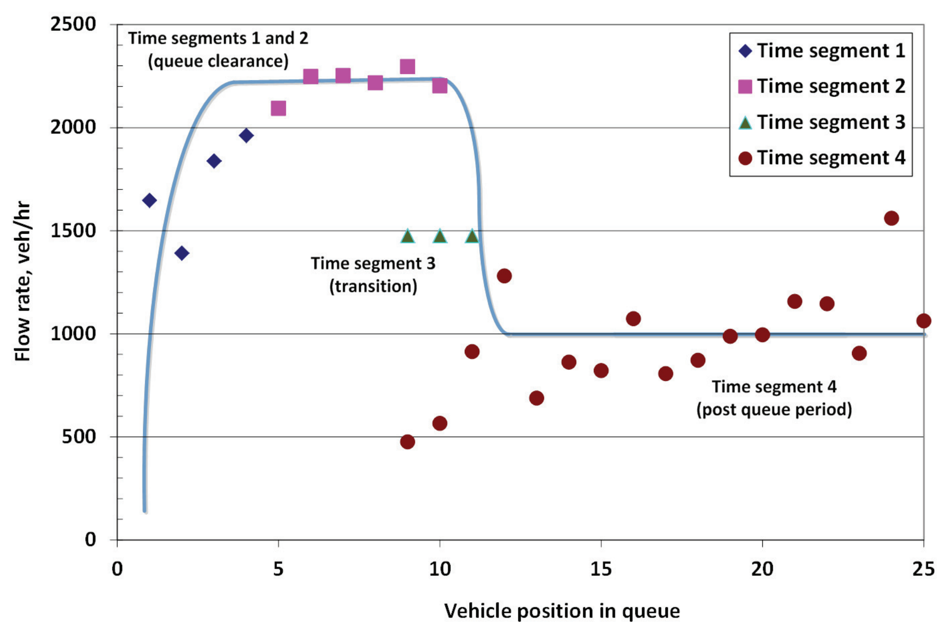
The reality is that queue clearance is a messy process with field measured values of headway and flow rate varying significantly about the theoretical values shown in Figure 108. Drivers respond to the change to green at different rates, and once they begin to move, they establish varying following distances behind the preceding vehicle. Since our desire is to extend the green only as long as a queue is present, and to terminate the green when the queue has cleared, we need to better understand the stochastic nature of this process. This involves understanding the headway, flow rate, and unoccupancy time distributions during queue clearance (time segments 1 and 2), the transition period (time segment 3) and the post-queue period (time segment 4).

To gain a perspective on this problem, the results from a set of simulation runs using the VISSIM microscopic simulation model are presented here (Kyte, Urbanik, & Amin, 2007). For these simulations, the queue at the beginning of green ranged from eight to ten vehicles. The traffic control was set to fixed time so that additional vehicles would be served after this initial queue had cleared and a comparison between headways of vehicles in the departing queue and during the post queue period could be made.

Figure 111a shows the mean headways measured for the first 25 vehicles passing the stop bar at the beginning of green. The dashed line represents the theoretical departure headways shown previously in Figure 109b. The simulation data are shown varying about this theoretical line, the kind of stochastic variation that we expect to see in the field. The headways measured during queue clearance vary in a narrow range about the theoretical line. However, the headways during the post queue period have a much wider variation with some almost as low as values that were measured during queue clearance. Figure 111b shows the mean flow rates for these same positions in queue, based on the headways shown in Figure 111a.



a. Headway data



b. Flow rate data

Figure 111. Simulated data (and theoretical line) by vehicle position 111

This stochastic variation has two implications in the selection of the MAH. First, even headways in the departing queue have some variation reflecting differences in driver characteristics. Second, headways in the post queue period may be similar to those during the queue clearance period, thus making it difficult to determine, just based on headway values, which period you are observing.

Multiple lane approaches

How does the problem of setting the passage time change if there is more than one lane on the intersection approach? Often the detection scheme is such that the control system only knows that a call has been received on an approach, not which lane the call comes from. This means that the headway distribution that the traffic control system "sees" is the combined distribution of both lanes. This situation is illustrated in Figure 112 which shows an example of the departure of vehicles over a 30 second period for two lanes individually (Lane 1 and Lane 2) and then taken together (both lanes). In the figure, the headway between any two vehicles is represented by the horizontal distance between the points representing the two vehicles. Two example headways are noted in lanes 1 and 2. For lane 1, the headway is 8.0 seconds, while for lane 2 it is 3.0 seconds. However, if we measure the headways using vehicles from both lanes together (as shown for the three vehicles from lanes 1 and 2 "boxed" together), the consecutive headways would be 0.3 seconds and 2.7 seconds. The picture given with data combined from both lanes is a different one than the headways shown for lanes 1 and 2 separately. And, the conclusion would be different as well. The detection system would "see" three closely spaced vehicles (0.3 and 2.7 seconds) and conclude something different than if the headways are measured from each lane separately (8.0 and 3.0 seconds).

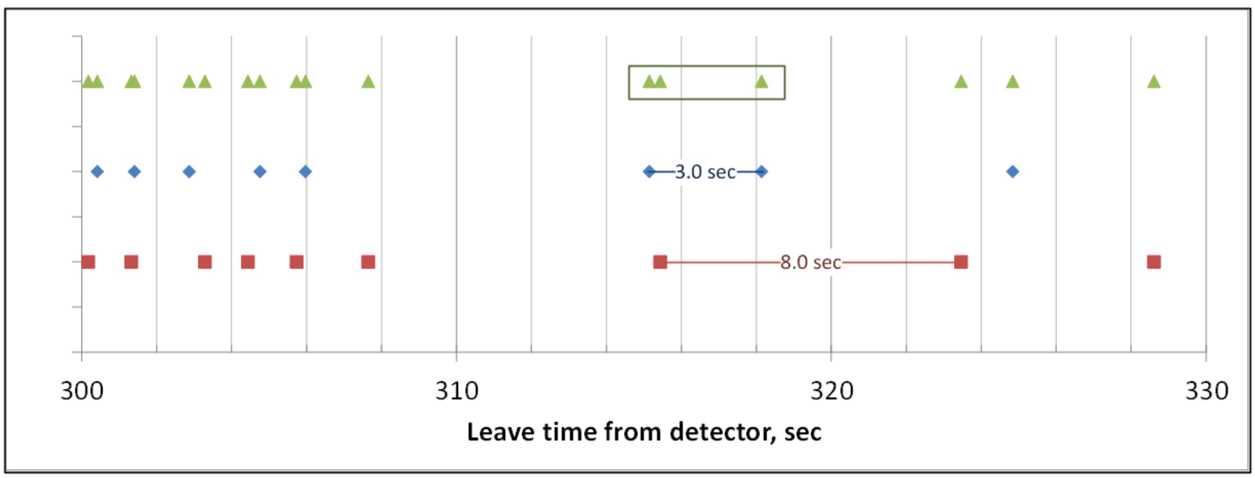


Figure 112. "Leave time" for lane 1, lane 2, and both lanes together

Let's now consider the headway density and cumulative density functions in a departing queue measured for one lane separately and for two lanes combined for an intersection approach, as shown in Figure 113 and Figure 114. While the density functions look similar, the two-lane data are shifted to the left, compared to the one-lane data. The mean value for the one lane data is 1.73 seconds and 0.84 seconds for the two lane (combined) data.

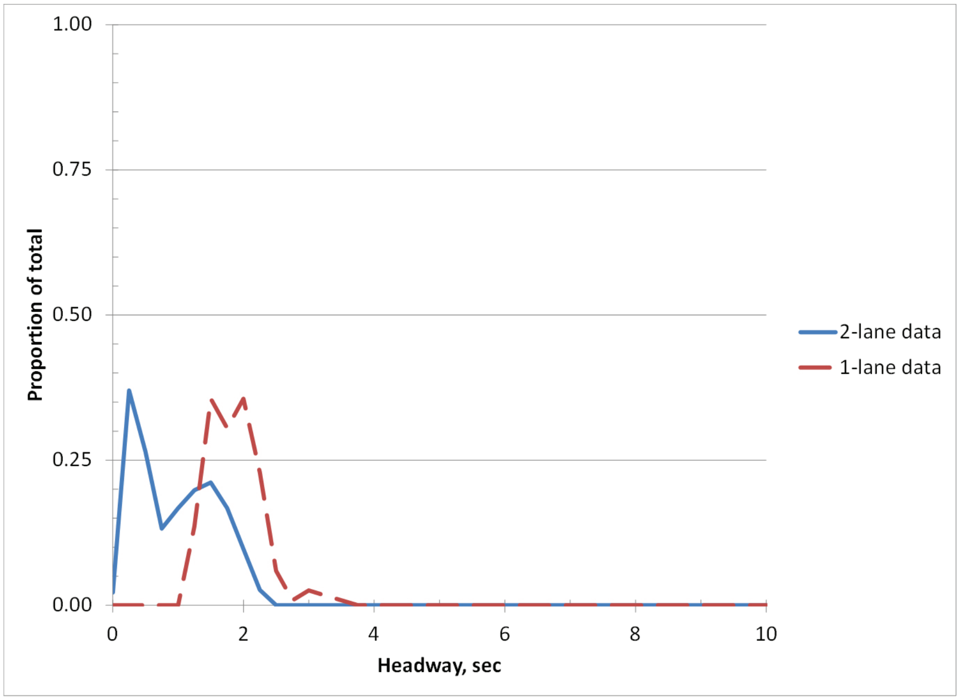


Figure 113. Headway density function, one lane and two lane data

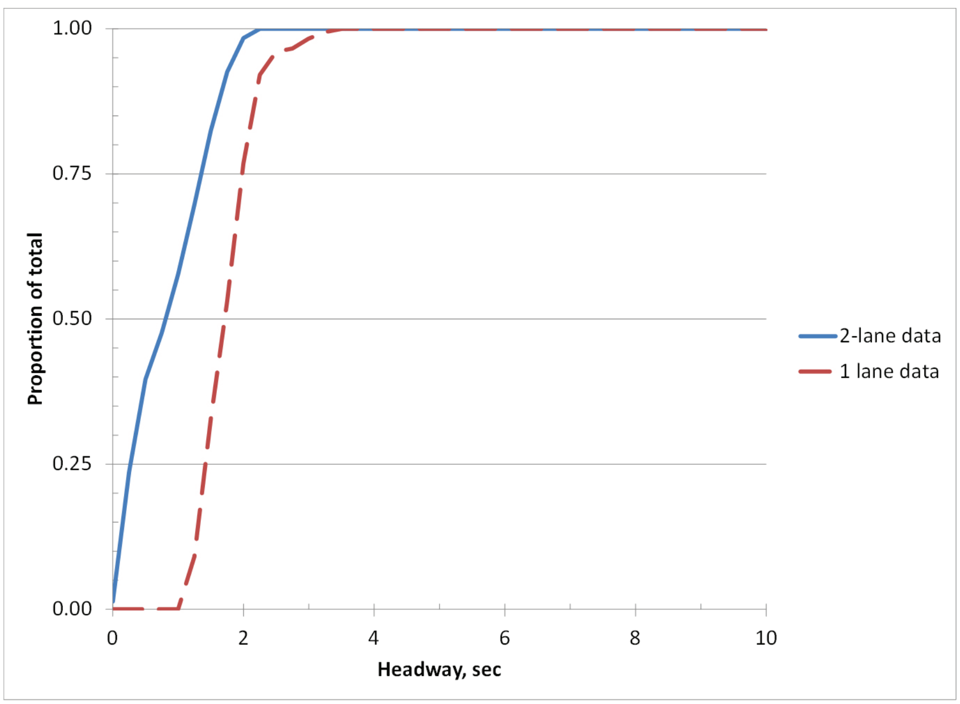


Figure 114. Cumulative headway density function, one lane and two lane data

What are the implications of these headway distribution differences when it comes to setting the passage time? If we assumed a vehicle speed of 25 miles per hour (36.75 feet per second), an average vehicle length of 20 feet and a detection zone length of 22 feet, we could calculate the resulting unoccupancy time for both the one lane and two lane conditions for a given value of headway. Let's consider three cases, in which 99, 95, and 90 percent of the vehicles in the queue would be served by a given MAH. The resulting calculations for the unoccupancy time (and thus the passage time) are shown in Table 15, based on the equation that we considered earlier for the calculation of the unoccupancy time.

Percentile	1-Lane		2-Lane	
	Headway (sec)	Unoccupancy time (sec)	Headway (sec)	Unoccupancy time (sec)
99	3.1	2.0	2.1	1.0
95	2.4	1.3	1.8	0.7
90	2.2	1.1	1.7	0.6

Table 15. Maximum allowable headways and unoccupancy times for 1-Lane and 2-Lane conditions

This example shows that the unoccupancy times (and thus the passage times) are from 0.5 to 1.0 second lower for the 2-lane case than for the 1-lane case. The clear implication is that the passage time for a 2-lane approach should be lower than for a 1-lane approach if we are to achieve the same efficiency in signal timing and meet our objective of providing sufficient green time to serve a clearing queue but not vehicles that arrive after the queue has cleared.

Conclusion

The minimum green time establishes the minimum time that the green will be displayed for a phase. The passage time parameter determines how long the green will be extended after the minimum green timer has expired and is directly related to the MAH and the length of the detection zone. The stochastic variation of headways in the vehicle stream means that the challenge in selecting the MAH (and thus the passage time) is to balance two risks. The first risk (if the MAH is too short) is that the phase will be terminated too early if the queue is still clearing. The second risk (if the MAH is too long) is that the phase will be extended past the time that the queue has cleared. Selecting the MAH, and then the passage time, is the balance in risks that the transportation engineer must determine. Finally, we need to consider shorter passage times for a 2-lane approach than the value we would consider for a one-lane approach. The activities to follow will give you specific experiences in dealing with each of these issues.