PREDICTING THE PERFORMANCE OF AUTOMOBILE TRAFFIC ON URBAN STREETS

FINAL REPORT

Prepared For
National Cooperative Highway Research Program
Transportation Research Board of
The National Academies

Texas Transportation Institute
Texas A&M University

In Association with:
Kittelson & Associates, Inc.
Purdue University

January 2008
ACKNOWLEDGMENT OF SPONSORSHIP

This work was sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program which is administered by the Transportation Research Board of the National Academies.

DISCLAIMER

This is an uncorrected draft as submitted by the research agency. The opinions and conclusions expressed or implied in the report are those of the research agency. They are not necessarily those of the Transportation Research Board, the National Academies, or the program sponsors.
Project 3-79

Predicting the Performance of Automobile Traffic on Urban Streets

JAMES A. BONNESON
MICHAEL P. PRATT
Texas Transportation Institute
College Station, Texas

MARK A. VANDEHEY
Kittelson & Associates, Inc.
Portland, Oregon

Subject Areas
Highway and Facility Design
Highway Operations, Capacity and Traffic Control

Research Sponsored by the American Association of State Highway and Transportation Officials in Cooperation with the Federal Highway Administration

Transportation Research Board of the National Academies
Washington, D.C.

January 2008
# CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIST OF FIGURES</td>
<td>vi</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>vii</td>
</tr>
<tr>
<td>ACKNOWLEDGMENTS</td>
<td>viii</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>ix</td>
</tr>
<tr>
<td>EXECUTIVE SUMMARY</td>
<td>1</td>
</tr>
<tr>
<td>CHAPTER 1 Introduction and Research Approach</td>
<td>5</td>
</tr>
<tr>
<td>Problem Statement</td>
<td>5</td>
</tr>
<tr>
<td>Research Objective</td>
<td>5</td>
</tr>
<tr>
<td>Research Scope</td>
<td>5</td>
</tr>
<tr>
<td>Research Approach</td>
<td>6</td>
</tr>
<tr>
<td>CHAPTER 2 Findings</td>
<td>7</td>
</tr>
<tr>
<td>Highway Capacity Manual Methodology</td>
<td>7</td>
</tr>
<tr>
<td>Alternative Performance Prediction Procedures</td>
<td>12</td>
</tr>
<tr>
<td>Evaluation of Performance Prediction Procedures</td>
<td>25</td>
</tr>
<tr>
<td>CHAPTER 3 Interpretation, Appraisal, and Applications</td>
<td>29</td>
</tr>
<tr>
<td>Proposed Methodology Framework</td>
<td>29</td>
</tr>
<tr>
<td>Proposed Procedures</td>
<td>31</td>
</tr>
<tr>
<td>Verification</td>
<td>37</td>
</tr>
<tr>
<td>CHAPTER 4 Conclusions and Recommendations</td>
<td>41</td>
</tr>
<tr>
<td>Conclusions</td>
<td>41</td>
</tr>
<tr>
<td>Recommendations</td>
<td>42</td>
</tr>
<tr>
<td>REFERENCES</td>
<td>45</td>
</tr>
<tr>
<td>APPENDIX A Procedure for Estimating Delay Due to Turning Vehicles</td>
<td>A-1</td>
</tr>
<tr>
<td>APPENDIX B Procedure for Estimating Running Time</td>
<td>B-1</td>
</tr>
<tr>
<td>APPENDIX C Procedure for Estimating Arrival Flow Profile</td>
<td>C-1</td>
</tr>
<tr>
<td>APPENDIX D Procedure for Estimating Coordinated-Actuated Phase Duration</td>
<td>D-1</td>
</tr>
<tr>
<td>APPENDIX E Procedure for Estimating Stop Rate at a Signalized Intersection</td>
<td>E-1</td>
</tr>
<tr>
<td>APPENDIX F Procedure for Addressing Capacity Constraints</td>
<td>F-1</td>
</tr>
<tr>
<td>APPENDIX G Verification of Methodology and Engine</td>
<td>G-1</td>
</tr>
<tr>
<td>APPENDIX H Proposed Text for the <em>Highway Capacity Manual</em></td>
<td>H-1</td>
</tr>
</tbody>
</table>
### LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Urban street analysis segment</td>
<td>8</td>
</tr>
<tr>
<td>2.</td>
<td>Comparison of guidance relating free-flow speed to speed limit</td>
<td>15</td>
</tr>
<tr>
<td>3.</td>
<td>Relationship between access point density, speed limit, and free-flow speed</td>
<td>17</td>
</tr>
<tr>
<td>4.</td>
<td>Influence of segment length on cruise speed</td>
<td>18</td>
</tr>
<tr>
<td>5.</td>
<td>Relationship between segment length, volume, and speed</td>
<td>18</td>
</tr>
<tr>
<td>6.</td>
<td>Running speed as a function of flow rate</td>
<td>20</td>
</tr>
<tr>
<td>7.</td>
<td>Residual start-up lost time as a function of segment length</td>
<td>21</td>
</tr>
<tr>
<td>8.</td>
<td>Effect of offset and platoon dispersion on delay and platoon ratio</td>
<td>23</td>
</tr>
<tr>
<td>9.</td>
<td>Proposed urban street methodology framework</td>
<td>30</td>
</tr>
<tr>
<td>10.</td>
<td>Arrival flow profile prediction</td>
<td>34</td>
</tr>
<tr>
<td>11.</td>
<td>Time elements influencing non-coordinated phase duration</td>
<td>36</td>
</tr>
<tr>
<td>12.</td>
<td>Illustration of vehicles stopping on an intersection approach</td>
<td>37</td>
</tr>
<tr>
<td>13.</td>
<td>Comparison of performance measure predictions</td>
<td>38</td>
</tr>
</tbody>
</table>
LIST OF TABLES

1. Urban street class ........................................................... 8
2. Functional and design categories ........................................ 9
3. Urban street level of service thresholds ............................... 10
4. Segment running time rate ................................................. 11
5. Base free-flow speed adjustment factors for multilane highways 15
6. Procedures developed for the proposed methodology ............... 32
ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 3-79 by the Texas Transportation Institute (TTI); Kittelson & Associates, Inc. (KAI); and Purdue University. Dr. James Bonneson of TTI served as the principal investigator for the project and supervised the research conducted by TTI. Mr. Mark Vandehey supervised the research conducted by KAI and Dr. Darcy Bullock supervised the research conducted by Purdue University. The research supervisors were assisted by the following individuals:

- Mr. Scott Beaird Kittelson & Associates, Inc.
- Mr. Peter Koonce Kittelson & Associates, Inc.
- Mr. Kevin Lee Kittelson & Associates, Inc.
- Mr. Michael P. Pratt Texas Transportation Institute
- Dr. Karl Zimmerman Valparaiso University

These individuals made important contributions to the research project through their participation in its various research tasks.
Chapter 15 of the *Highway Capacity Manual (HCM)* describes a methodology for predicting the performance of an urban street in terms of travel speed. However, urban street operations continue to increase in complexity and users of the methodology continue to identify the need for more sophisticated procedures. The objective of this research was to revise the *HCM* methodology such that it can be used to more accurately predict the performance of automobile traffic on urban streets. To satisfy this objective, a procedure for predicting the free-flow speed and running time associated with an urban street was developed and calibrated. The proposed procedure was described in the form of recommended content for the next edition of the *HCM*. 
EXECUTIVE SUMMARY

INTRODUCTION

Chapter 15 of the *Highway Capacity Manual (HCM)* describes a methodology for predicting the performance of an urban street in terms of travel speed. However, urban street operations continue to increase in complexity and users of the methodology continue to identify the need for more sophisticated procedures. The *HCM* methodology for predicting travel speeds has weaknesses, particularly in the determination of the free-flow speed and failure to fully account for some factors that influence travel speed, including arterial traffic volume, signal offset, access point density, cross-section design, arterial weaving, and platoon dispersion.

The objective of this research was to revise the *HCM* methodology such that it can be used to more accurately predict the performance of automobile traffic on urban streets. To satisfy this objective, a procedure for predicting the free-flow speed and running time associated with an urban street was developed and calibrated. The proposed procedure was described in the form of recommended content for the next edition of the *HCM*.

FINDINGS

This section summarizes the findings from an evaluation of alternative performance prediction procedures. The focus of this evaluation is on procedures that predict measures that describe the operational performance of automobile traffic flow on urban streets. These measures include running time, delay, and stop rate. The objective is to identify procedures that should be considered for inclusion in the methodology described in Chapter 15 of the *HCM*. These findings were used to define a plan for developing and calibrating the needed procedures.

The focus of the evaluation is on two procedures in the *HCM* methodology. One procedure is used to estimate running time and the other is used to estimate signal control delay. The next section summarizes an evaluation of the running time prediction procedure. The subsequent section evaluates the signal control delay prediction procedure. In both parts, the discussion is focused on factors that (1) have been documented as having an influence on urban street traffic operations, and (2) may not be adequately accounted for in the *HCM* methodology.

Running Time

There are several factors that influence the running time incurred by motorists traveling along an urban street. They tend to slow (or delay) motorists by increasing running time beyond that required when traveling at the free-flow speed. The amount of delay associated with these factors tends to be small, when compared to the delay incurred at a signalized intersection. However, it can be significant in some specific situations and should be part of a detailed evaluation of segment running time. These factors include:

1. influence of segment length on free-flow speed,
2. delay due to vehicles turning right from a through lane,
3. delay due to vehicles turning left from a through lane,  
4. factors influencing free-flow speed (e.g., access point density, lane width, lateral clearance),  
5. delay due to proximity of other vehicles (i.e., effect of traffic density on speed), and  
6. delay due to parking maneuvers.

This list ranks the factors in terms of their associated delay influence and the frequency with which they are likely to be encountered on most urban streets (the most important factor is listed first).

**Signal Control Delay**

There are several controller functions and flow processes identified in the literature that have an influence on control delay at a coordinated-actuated intersection. Based on this review, the following list ranks the factors that significantly influence control delay and which could be more accurately modeled in the **HCM** methodology:

1. basic signal coordination (i.e., platoon dispersion),  
2. green interval timing (i.e., average phase duration),  
3. semi-actuated signal coordination (i.e., signal offset relationship), and  
4. upstream signal metering and queue spillback.

This list ranks the factors in terms of their associated delay influence and the frequency with which they are likely to be encountered on most urban streets (the most important factor is listed first).

**Stop Rate**

Stops are generally expected by motorists when arriving at an intersection as a minor movement (e.g., a turn movement or a through movement on the minor street). However, through drivers do not expect to stop when traveling along a major street. Their expectation is that the signals will be coordinated to some degree such that they can arrive at each signal in succession while it is displaying a green indication for the through movement. For this reason, stop rate is a useful performance measure for evaluating coordinated signal systems. Unfortunately, the **HCM** methodology does not currently have a procedure for estimating stop rate.

**CONCLUSIONS**

Several procedures were developed for the **HCM** urban street performance evaluation methodology. The proposed procedures are intended to improve the accuracy of the estimated running time and control delay. These procedures are identified in the list below:

- delay due to turning vehicles,  
- running time (including free-flow speed),  
- arrival flow profile,  
- actuated phase duration,  
- stop rate at a signalized intersection, and  
- capacity constraints.
The stop rate prediction procedure was developed to extend the range of performance measures predicted by the HCM methodology. Each of these procedures is described in an appendix.

The accuracy of the proposed procedures was evaluated by comparing the predicted performance measures with those obtained from a traffic simulation model. The findings from this analysis indicate that the predicted delay from the proposed procedures is within one or two seconds of that obtained from the simulation model. A similar good fit was found when comparing the predicted stop rate with that obtained from the simulation model. The analysis also indicated that proposed procedures yield a reasonably good estimate of the simulated travel speed.

**RECOMMENDATIONS**

The research conducted for this project has led to the formulation of several recommendations. These recommendations are described in the following paragraphs.

It is recommended that the proposed methodology described in Appendix H be considered as a replacement for the methodology in Chapter 15 of the HCM.

Even though the proposed methodology has addressed the most significant limitations of the methodology in Chapter 15 of the HCM, several additional limitations remain that should be the subject of future research. These limitations are summarized in the following paragraphs.

With regard to running time, research is needed to evaluate the effect of on-street parking activity, trucks, and grade on running time. If these factors are found to have some influence, then the research should develop models for quantifying their impact to segment running time.

With regard to actuated controller operation, research is needed to extend the actuated phase duration prediction procedure to account for pedestrian detection and right-turn phase overlap.

With regard to stop rate prediction, research is needed to evaluate the effect of mid-segment turns on through vehicle stop rate. If turns into a mid-segment access point are found to have a significant effect on running time, then this research should develop a model for quantifying this impact.

With regard to capacity constraints, research is needed to develop a procedure for estimating phase capacity when frequent cyclic spillback occurs. This type of spillback occurs when the maximum back of queue from the downstream signal backs into the subject intersection briefly each signal cycle.

Finally, research is needed on the operational effects of added lanes on the approach and departure legs of an intersection (also called “flared” approaches). This research should specifically examine the effects of these lanes on lane utilization and on segment operation.
CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

PROBLEM STATEMENT

Chapter 15 of the *Highway Capacity Manual (HCM)* (1) describes a methodology for predicting the performance of an urban street in terms of travel speed. However, urban street operations continue to increase in complexity and users of the methodology continue to identify the need for more sophisticated procedures.

One weakness of the *HCM* methodology is its procedure for estimating travel speed. This procedure requires an estimate of the free-flow speed for the facility. To assist in the determination of free-flow speed, the *HCM* offers two choices: (1) review the qualitative descriptions of four street classes, find the best match to the subject facility, and use the associated default free-flow speed; or (2) measure the free-flow speed in the field. The former approach lacks precision and the latter approach is not practical for routine evaluations because it requires as much effort as the direct field measurement of travel speed.

A second weakness of the *HCM* methodology is its relatively simple procedure for estimating running time. This time is based on the following factors: distance between signalized intersections, street class, and free-flow speed. Unfortunately, this list of factors that influence running time is not complete. Other factors that should be evaluated in the research include traffic volume, signal offset, access point density, speed limit, and cross section (e.g., lane width, lateral clearance, median type).

RESEARCH OBJECTIVE

The objective of this research is to revise the *HCM* methodology such that it can be used to more accurately predict the performance of automobile traffic on urban streets. To satisfy this objective, the following goals were established:

- Develop and calibrate a procedure for predicting the free-flow speed and running time associated with an urban street and eliminate the known inaccuracies associated with the existing *HCM* methodology.

- Facilitate implementation of the research findings by developing recommended content for the next edition of the *HCM*.

RESEARCH SCOPE

This report uses the term “urban street” although the research results are applicable to a broad range of interrupted-flow arterial and collector roads in urban, suburban, and rural settings. The research products focus on the operational performance of the automobile traffic stream (which may include a mixture of passenger cars, trucks, and buses). The procedures described in this report can be described as deterministic because they focus on quantifying an expected value for each predicted
measure based on an assumed steady flow condition. The methodology is envisioned to have a form that is perceived by practitioners as cost-effective for a range of applications that are not currently served by area-wide planning models and simulation-based operations models.

**RESEARCH APPROACH**

The research approach was directed toward the development of a methodology for predicting the performance of urban street facilities, including the intersections and individual segments of which they are comprised. To the extent possible, the structure of the existing HCM procedure was retained but its component predictive procedures were refined or supplemented with new procedures. The work tasks that were undertaken to develop the new methodology include:

- Task B.1: Review and Critique Literature on Urban Street Operations
- Task B.2: Identify Performance Measures and Assess Prediction Methods
- Task B.3: Develop Analytic Methods and Prepare Data Collection Plan
- Task B.4: Submit Interim Report
- Task B.5: Execute Data Collection Plan
- Task B.6: Calibrate Models and Develop Analysis Procedure
- Task B.7: Develop Computational Engine
- Task B.8: Prepare Draft Text for the *HCM*
- Task B.9: Submit Final Report

The main products of the research are: (1) draft text for the *HCM* that reflects the changes necessary to implement the proposed performance prediction methodology developed in this research, and (2) a computational engine that automates the proposed methodology.
CHAPTER 2

FINDINGS

This chapter documents the findings related to a review of the HCM methodology and its delay and running time prediction procedures. The discussion focuses on the component models that comprise each procedure. Also documented is a review and evaluation of alternative component models. The evaluation consists of an assessment of the alternative model’s ability to improve an HCM procedure. The last part of this chapter identifies procedures that should be considered for inclusion in the methodology described in Chapter 15 of the HCM.

HIGHWAY CAPACITY MANUAL METHODOLOGY

This part of the chapter provides a brief overview of the urban streets performance evaluation methodology described Chapter 15 of the HCM. The topics addressed include:

- Scope and Limitations
- Urban Street Class
- Level of Service
- Analysis Steps

The discussion associated with each topic is described in a separate section.

Scope and Limitations

The HCM methodology is based on the evaluation of one direction of traffic flow along a street segment. The segment can be one of several that compose an urban street facility. In fact, if each segment is individually evaluated, the performance of the entire facility can be computed by aggregating the results for each segment.

The methodology includes two key procedures. One procedure estimates the delay due to the signal at the end of the segment. The second procedure estimates the running time along the segment. Both of these measures are computed for the through traffic movement in the subject direction of travel.

A street segment is defined in the HCM as that portion of an urban street facility that is bounded on each end by a signalized intersection and includes all traffic lanes serving one direction of travel. The segment includes the through approach to the downstream signalized intersection. This concept is illustrated in Figure 1, where the shaded area shown represents the street segment that is the subject of analysis.

The HCM methodology is used to analyze one or more segments on an urban street facility, provided that the facility has a minimum length of 2 mi (1 mi in downtown area) and a signal spacing of 2 mi or less. The methodology is applicable to segments with a minimum length that
ranges from 440 ft for urban minor arterial streets to 0.5 mi for high-speed principal arterial streets. Finally, the methodology is applicable to streets with a speed limit ranging from 25 to 55 mph.

![Urban street analysis segment.](image)

The HCM methodology does not directly account for the following factors that can influence travel speed:

- presence of on-street parking;
- access point density;
- lane additions and drops at intersections;
- grade;
- capacity constraints between signals;
- median treatment;
- turn movements that exceed 20 percent;
- queue spillback; and
- cross-street congestion.

**Urban Street Class**

The HCM methodology requires identification of urban street class to determine level of service. Four classes are defined (i.e., I, II, III, and IV). Determination of the correct class designation requires determination of the street’s functional and design categories. This determination is based on the analyst’s subjective evaluation of a series of criteria for which typical values or conditions are stated. The relationship between class and category is shown in HCM Exhibit 10-3; the criteria used to determine functional and design categories are listed in HCM Exhibit 10-4. Both exhibits are shown herein as Tables 1 and 2, respectively.

<table>
<thead>
<tr>
<th>Design Category</th>
<th>Functional Category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Principal Arterial</td>
</tr>
<tr>
<td>High-Speed</td>
<td>I</td>
</tr>
<tr>
<td>Suburban</td>
<td>II</td>
</tr>
<tr>
<td>Intermediate</td>
<td>II</td>
</tr>
<tr>
<td>Urban</td>
<td>III or IV</td>
</tr>
</tbody>
</table>

**Figure 1. Urban street analysis segment.**

The HCM methodology does not directly account for the following factors that can influence travel speed:

- presence of on-street parking;
- access point density;
- lane additions and drops at intersections;
- grade;
- capacity constraints between signals;
- median treatment;
- turn movements that exceed 20 percent;
- queue spillback; and
- cross-street congestion.
TABLE 2 Functional and design categories

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Functional Category</th>
<th>Principal Arterial</th>
<th>Minor Arterial</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobility function</td>
<td>Very important</td>
<td>Important</td>
<td></td>
</tr>
<tr>
<td>Access function</td>
<td>Very minor</td>
<td>Substantial</td>
<td></td>
</tr>
<tr>
<td>Points connected</td>
<td>Freeways, important activity centers, major traffic generators</td>
<td>Principal arterials</td>
<td></td>
</tr>
<tr>
<td>Predominant trips served</td>
<td>Relatively long trips between major points and through-trips entering, leaving, and passing through the city</td>
<td>Trips of moderate length within relatively small geographical areas</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Design Category</th>
<th>High-speed</th>
<th>Suburban</th>
<th>Intermediate</th>
<th>Urban</th>
</tr>
</thead>
<tbody>
<tr>
<td>Access density</td>
<td>Very low</td>
<td>Low</td>
<td>Moderate</td>
<td>High</td>
<td></td>
</tr>
<tr>
<td>Arterial type</td>
<td>Multilane divided; undivided or two lane with shoulders</td>
<td>Multilane divided; undivided or two lane with shoulders</td>
<td>Multilane divided or undivided; one-way, two-lane</td>
<td>Undivided one-way, two-way, two or more lanes</td>
<td></td>
</tr>
<tr>
<td>On-street parking</td>
<td>None</td>
<td>None</td>
<td>Some</td>
<td>Significant</td>
<td></td>
</tr>
<tr>
<td>Separate left-turn lanes</td>
<td>Yes</td>
<td>Yes</td>
<td>Usually</td>
<td>Some</td>
<td></td>
</tr>
<tr>
<td>Signal density</td>
<td>0.5 to 2 sig/mi</td>
<td>1 to 5 sig/mi</td>
<td>4 to 10 sig/mi</td>
<td>6 to 12 sig/mi</td>
<td></td>
</tr>
<tr>
<td>Speed limit</td>
<td>45 to 55 mph</td>
<td>40 to 45 mph</td>
<td>30 to 40 mph</td>
<td>25 to 35 mph</td>
<td></td>
</tr>
<tr>
<td>Pedestrian activity</td>
<td>Very little</td>
<td>Little</td>
<td>Some</td>
<td>Usually</td>
<td></td>
</tr>
<tr>
<td>Development density</td>
<td>Low</td>
<td>Low to medium</td>
<td>Medium to moderate</td>
<td>High</td>
<td></td>
</tr>
</tbody>
</table>

Level of Service

Urban street level of service (LOS) is based on average through-vehicle travel speed. Through vehicles are defined as those vehicles traveling as a through movement at both the upstream and subject signalized intersections. LOS thresholds for the four urban street classes are provided in Exhibit 15-2 of the HCM; this exhibit is reproduced as Table 3. An examination of the threshold speeds in this table indicates that the lower limits of LOS A, B, C, D, and E are about 85, 67, 50, 40, and 30 percent of the free-flow speed.

Analysis Steps

This section describes the four analysis steps that comprise the HCM methodology. The first step is to identify the input values. The second step is to determine the segment running time. The third step is to determine the through lane group delay. The last step is to determine the travel speed and level of service. Considerations associated with each step are described in the following subsections.
### TABLE 3 Urban street level-of-service thresholds

<table>
<thead>
<tr>
<th>Urban Street Class</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range of free-flow speeds, mph</td>
<td>45 to 55</td>
<td>35 to 45</td>
<td>30 to 35</td>
<td>25 to 35</td>
</tr>
<tr>
<td>Typical free-flow speed, mph</td>
<td>50</td>
<td>40</td>
<td>35</td>
<td>30</td>
</tr>
<tr>
<td>Level of Service</td>
<td>Average Travel Speed, mph</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>&gt; 42</td>
<td>&gt; 35</td>
<td>&gt; 30</td>
<td>&gt; 25</td>
</tr>
<tr>
<td>B</td>
<td>&gt; 34 - 42</td>
<td>&gt; 28 - 35</td>
<td>&gt; 24 - 30</td>
<td>&gt; 19 - 25</td>
</tr>
<tr>
<td>C</td>
<td>&gt; 27 - 34</td>
<td>&gt; 22 - 28</td>
<td>&gt; 18 - 24</td>
<td>&gt; 13 - 19</td>
</tr>
<tr>
<td>D</td>
<td>&gt; 21 - 27</td>
<td>&gt; 17 - 22</td>
<td>&gt; 14 - 18</td>
<td>&gt; 9 - 13</td>
</tr>
<tr>
<td>E</td>
<td>&gt; 16 - 21</td>
<td>&gt; 13 - 17</td>
<td>&gt; 10 - 14</td>
<td>&gt; 7 - 9</td>
</tr>
<tr>
<td>F</td>
<td>≤ 16</td>
<td>≤ 13</td>
<td>≤ 10</td>
<td>≤ 7</td>
</tr>
</tbody>
</table>

### Identify Inputs

The *HCM* procedure requires the following input data for the through lane group of each analysis segment:

- Cycle length
- Capacity
- Segment length
- Urban street class
- Green-to-cycle-length ratio
- Platoon arrival type (progression quality)
- Initial queue length
- Free-flow speed

Capacity is often estimated using the saturation flow rate procedure described in Chapter 16 of the *HCM*.

### Determine Running Time

*HCM* Exhibit 15-3 (reproduced as Table 4) is used to estimate the running time rate for the subject segment. As indicated by the table values, shorter segments are associated with a longer running time rate. This trend implies that segment length has some influence on speed, with lower running speeds found on shorter segments. The effect of traffic volume, signal offset, access point density, speed limit, and cross section on running time rate is not reflected in Table 4.
TABLE 4  Segment running time rate

<table>
<thead>
<tr>
<th>Urban Street Class</th>
<th>Free-Flow Speed, mph</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>55</td>
<td>50</td>
<td>45</td>
<td>45</td>
<td>40</td>
</tr>
<tr>
<td>Segment Length, mi</td>
<td>Running Time Rate ($t_R$), s/mi</td>
<td>0.05</td>
<td>0.10</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>0.05</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>0.10</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>0.15</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>0.20</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>0.25</td>
<td>97</td>
<td>100</td>
<td>104</td>
<td>104</td>
<td>120</td>
</tr>
<tr>
<td>0.30</td>
<td>92</td>
<td>95</td>
<td>99</td>
<td>99</td>
<td>102</td>
</tr>
<tr>
<td>0.40</td>
<td>82</td>
<td>86</td>
<td>94</td>
<td>94</td>
<td>96</td>
</tr>
<tr>
<td>0.50</td>
<td>73</td>
<td>78</td>
<td>88</td>
<td>88</td>
<td>93</td>
</tr>
<tr>
<td>1.00</td>
<td>65</td>
<td>72</td>
<td>80</td>
<td>80</td>
<td>90</td>
</tr>
</tbody>
</table>

Note:
“--”: Unusual combination of segment length and free-flow speed. See footnotes to Exhibit 15-3 of the HCM for additional guidance regarding the estimation of running time rate.

Determine Through-Lane-Group Delay

The HCM methodology offers the following equation for estimating the delay to the through lane group:

$$d = d_1 (PF) + d_2 + d_3$$  \hspace{1cm} (1)

where,

$$d_1 = \frac{0.5 C (1 - g/C)^2}{1 - \min(1, X) g/C}$$  \hspace{1cm} (2)

$$d_2 = 900 T \left[ (X - 1) + \sqrt{(X - 1)^2 + \frac{8 k I X}{C T}} \right]$$  \hspace{1cm} (3)

with,

$d_1$ = control delay to the through-lane-group, s/veh;
$d_2$ = uniform delay, s/veh;
$d_3$ = incremental delay, s/veh;
$PF$ = progression adjustment factor (see HCM Exhibit 15-5);
$X$ = volume to capacity ratio for the through lane group;
$C$ = cycle length, s;
$c$ = capacity of the lane group, veh/h;
$g$ = effective green time for the through lane group, s;
$T$ = duration of analysis period, h;
$k$ = incremental delay adjustment for actuated control; and
$I$ = incremental delay adjustment for filtering by upstream signal.
The progression adjustment factor $PF$ is obtained from HCM Exhibit 15-5. This factor is used in Equation 1 when arrivals to the intersection are influenced by upstream signals. The magnitude of this adjustment factor is based on the green-to-cycle-length ratio and the estimated arrival type. Values can range from 0.0 to 2.5, but typically vary from 0.7 to 1.3. Arrival type is determined by the analyst based on a subjective estimate of progression quality.

The incremental delay adjustment factor for actuated control is used to account for the ability of the detection design to minimize green extension due to random arrivals. Values for this factor vary from 0.04 to 0.50, depending on the controller passage time and lane-group volume-to-capacity ratio. HCM Exhibit 16-13 lists the recommended factor values.

The incremental delay adjustment for filtering is used to account for the effect of the upstream signal on the randomness in arrivals to the subject lane group. An upstream signal tends to moderate the variation in arrivals on a cycle-by-cycle basis.

_Determine Travel Speed and Level of Service_

The average through travel speed on the subject segment is computed as:

$$S_T = \frac{3600 \times L}{5280 \times (T_R + d)}$$

where,

- $S_T$ = average travel speed of through vehicles in the segment, mph;
- $L$ = segment length, ft;
- $T_R$ = segment running time ($= t_R \times L / 5280$), s; and
- $t_R$ = running time rate along the segment (see Table 4), s/mi.

Table 3 is then consulted using the average travel speed from Equation 4 to estimate the level of service for the segment.

_ALTERNATIVE PERFORMANCE PREDICTION PROCEDURES_

As described in the previous part, the HCM methodology includes two key procedures. One procedure estimates the delay due to the signal at the end of the segment. The second procedure estimates the running time along the segment. Each procedure is composed of several “component” models. This part of the report examines the factors that influence running time and delay by posing an alternative formulation for each procedure. The focus of the examination is on the component models that comprise each procedure and the factor influences they represent.

**Running Time**

The factors that contribute to segment running time are the subject of this section. They are identified in the following equation:

$$T_R = T_s + T_{gl}f_v + \sum_{t=1}^{n} d_{ap,t} + d_{other}$$

(5)
where,
\[ TR = \text{segment running time, s;} \]
\[ Ts = \text{residual start-up lost time, s;} \]
\[ T_{ff} = \text{free-flow running time (}= 0.682 \frac{L}{S_f}), \text{s;} \]
\[ L = \text{segment length, ft;} \]
\[ S_f = \text{free-flow speed, mph;} \]
\[ f_v = \text{proximity adjustment factor;} \]
\[ d_{ap, i} = \text{delay due to left- or right-turns from the street into access point } i, \text{s/veh;} \]
\[ N_{ap} = \text{number of access points along the subject segment, approaches;} \]
\[ d_v = \text{delay due to the proximity of other vehicles, s/veh;} \]
\[ d_{other} = \text{delay due to other sources (e.g., curb parking, pedestrians, bicyclists, etc.), s/veh.} \]

Equation 5 consists of four running time components. Models for predicting the first three components are described in this section. The fourth component accounts for the many other factors that could cause a driver to incur delay when traveling along a street. For example, a vehicle that is completing a parallel parking maneuver may cause following vehicles to incur some delay. Also, vehicles yielding to pedestrians at a mid-segment crosswalk incur delay. Finally, bicyclists riding in the traffic lane or in an adjacent bike lane may directly, or indirectly, cause vehicular traffic to adopt a lower speed and incur additional delay.

**Free-Flow Speed**

Free-flow speed is defined in Chapter 10 of the *HCM* to be the speed that a through automobile driver travels under low-volume conditions along an urban street when all the signals are green for the entire trip. Low-volume is defined as 200 veh/h/ln or less. Any delay due to signals or interactions with other vehicles is excluded from this speed. Free-flow speed is an average speed, as opposed to the 85th percentile speed.

This subsection examines the factors that influence free-flow speed based on the findings from a review of the literature. These factors include: speed limit, access point density, and segment length.

**Influence of Speed Limit.** As described in the previous section, free-flow speed is a fundamental input variable for the *HCM* urban streets methodology. It can be based on direct field measurement or on a determination of the street’s class. This latter determination is based on a subjective evaluation of the criteria listed in Tables 1 and 2. The selected urban street class is then used with Table 3 to estimate the typical free-flow speed. A comparison of the speed limit ranges in Table 2 with the typical free-flow speeds in Table 3 indicates that speed limits in the range of 25 to 35 mph, 35 to 40 mph, 40 to 45 mph, and 45 to 55 mph coincide with free-flow speeds of 30, 35, 40, and 50 mph, respectively. This trend suggests that the free-flow speed is about equal to the speed limit.

Tarko and Sinha (2) examined speed data from 116 speed measurement stations on arterial highways in Indiana. Their analysis found that several factors were correlated with free-flow speed, they include: heavy-vehicle percentage, time of day (i.e., day, night), speed limit, land use (i.e.,
urban, rural), number of lanes, and road class (i.e., freeway, nonfreeway). They found that speeds were lower on facilities with more lanes. They also found that higher percentages of heavy vehicles during the day were correlated with lower speeds but the reverse was true during the nighttime. The regression equations they developed indicate that urban, four-lane, non-freeway roads with speed limits of 55 and 65 mph have free-flow speeds of 61.7 and 67.5 mph, respectively.

The Florida Department of Transportation (FDOT) (3) recommends that the free-flow speed can be estimated as being 5 mph faster than the posted speed limit when conducting arterial level-of-service analyses.

Chapter 21 of the HCM describes a model for computing the free-flow speed for multilane highways. This model is based on the following equation:

\[ S_f = S_b - f_{LW} - f_{LC} - f_M - f_A \]  

where,

- \( S_f \) = free-flow speed, mph;
- \( S_b \) = base free-flow speed (may use 47, 52, 55, 60 for speed limits of 40, 45, 50, 55, respectively), mph;
- \( f_{LW} \) = adjustment for lane width, mph;
- \( f_{LC} \) = adjustment for lateral clearance, mph;
- \( f_M \) = adjustment for median treatment, mph; and
- \( f_A \) = adjustment for access points, mph.

The adjustment factors used in Equation 6 indicate that lane width, lateral clearance, median type, and access point density can reduce the free-flow speed. Values for each adjustment factor are listed in Table 5.

In a recent examination of speed data at 12 speed measurement stations on 12 rural multilane highways, Dixon et al. (4) found that multilane highways with speed limits of 55 and 65 mph have free-flow speeds of 61.8 and 64.9 mph, respectively. These speeds compare very favorably with those found by Tarko and Sinha (2).

Dowling et al. (5) examined speed data from 10 speed measurement stations on four rural highways in three states. They developed the following relationship between free-flow speed and speed limit:

\[ S_f = 14 + 0.88 S_{pl} \]  

where,

- \( S_f \) = posted speed limit, mph.

The guidance provided in the HCM and by the aforementioned researchers indicates that speed limit is likely correlated with free-flow speed. The guidance offered is summarized in Figure 2. With the exception of the guidance provided in Chapter 10 of the HCM, there is a trend where the free-flow speed is 5 to 7 mph faster than the speed limit. However, it should be noted that both Tarko and Sinha (2) and Dixon et al. (4) found that free-flow speed is only slightly above the speed limit when the speed limit is 65 mph. In general, the trends attributed to FDOT, HCM
Chapter 21, and Dowling et al. in Figure 2 suggest that the speed limit is about equal to the 15\(^{th}\) percentile speed.

**TABLE 5** Base free-flow speed adjustment factors for multilane highways

<table>
<thead>
<tr>
<th>Adjustment for Lateral Clearance</th>
<th>Four-Lane Highways</th>
<th>Six-Lane Highways</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Clearance, ft</td>
<td>(f_{LC}, \text{mph})</td>
<td>Lateral Clearance, ft</td>
</tr>
<tr>
<td>12</td>
<td>0.0</td>
<td>12</td>
</tr>
<tr>
<td>10</td>
<td>0.4</td>
<td>10</td>
</tr>
<tr>
<td>8</td>
<td>0.9</td>
<td>8</td>
</tr>
<tr>
<td>6</td>
<td>1.3</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>1.8</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>3.6</td>
<td>2</td>
</tr>
<tr>
<td>0</td>
<td>5.4</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Adjustment for Lane Width</th>
<th>(f_{LW}, \text{mph})</th>
<th>Access Density, ap/(\text{mi})</th>
<th>(f_{A}, \text{mph})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane Width, ft</td>
<td></td>
<td>Access Density, ap/(\text{mi})</td>
<td>(f_{A}, \text{mph})</td>
</tr>
<tr>
<td>12</td>
<td>0.0</td>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>11</td>
<td>1.9</td>
<td>10</td>
<td>2.5</td>
</tr>
<tr>
<td>10</td>
<td>6.6</td>
<td>20</td>
<td>5.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Adjustment for Median Treatment</th>
<th>(f_{M}, \text{mph})</th>
<th>Median Type</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Median Type</td>
<td>(f_{M}, \text{mph})</td>
<td>(\geq 40)</td>
<td>10.0</td>
</tr>
<tr>
<td>Undivided</td>
<td>1.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Divided</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 2.** Comparison of guidance relating free-flow speed to speed limit.
The trends shown in Figure 2 that are attributed to FDOT, *HCM* Chapter 21, and Dowling et al. indicate that roadways with a speed limit that is 5 mph faster than other roadways will also have a free-flow speed that is about 5 mph faster. That is, they imply a one-to-one correlation between the two variables. However, numerous researchers that independently investigated the effect of a speed limit change on observed traffic speeds report that the correlation is not one-to-one \((6, 7, 8, 9)\). In fact, these researchers have consistently found that the average speed changes only 2 to 3 mph when the speed limit changes by 10 mph. This trend has been found on streets and highways, when the speed is increased or decreased by 5 or 10 mph. It implies that the slope of the three trend lines in Figure 2 is too steep.

**Influence of Access Point Density.** Data reported in the literature were used to examine the correlation between access point density and free-flow speed. The data reported by Bonneson and McCoy \((10)\) describe seven urban street segments with access point densities ranging from 15 to 80 access points per mile (points/mi) and speed limits ranging from 30 to 45 mph. The data reported by Dixon et al. \((4)\) describe 12 multilane highway segments with driveway densities ranging from 2 to 14 points/mi. The data reported by Fitzpatrick et al. \((11)\) represent 69 urban/suburban street segments and 9 rural highway segments with access point densities ranging from 2 to 14 points/mi. The cited access point densities are based on a count of access points on both sides of the street. Thus, the one-side densities reported by Dixon et al. were doubled for this analysis.

All total, data for 97 road segments (76 urban, 21 rural) were assembled for the purpose of evaluating the relationship between speed limit and access point density on free-flow speed. A regression analysis of the combined database revealed the following equation for estimating free-flow speed:

\[
S_f = 26.0 - 12.7 I_{local} - 7.7 I_{art-col}\ + 0.61 S_{pl} - 0.031 D_a \tag{8}
\]

where,  
\(I_{art-col}\) = indicator variable for road type (1.0 for urban arterial or collector street, 0.0 otherwise);  
\(I_{local}\) = indicator variable for road type (1.0 for urban local street, 0.0 otherwise); and  
\(D_a\) = access point density (total for both sides of road), points/mi.

The coefficient of determination \(R^2\) for this equation is 0.94 and the \(t\)-statistic for each regression coefficient is significant at the 99 percent level or higher. The regression coefficient of -0.031 for access density is smaller than the value of -0.125 suggested by the adjustment factors listed in Table 5 \((i.e., 0.125 = 2.5/10/2, \text{where the divisor of ‘2’ is used to adjust for a two-way total access density})\). The coefficient of 0.61 for speed limit is smaller than 1.0, as suggested by the FDOT trend in Figure 2, but it larger than 0.2 to 0.3 suggested by the aforementioned research focused solely on the effect of speed limit change.

The fit of Equation 8 to the data is shown in Figure 3. Each data point shown represents the free-flow speed for all street or road segments having the corresponding functional classification and posted speed limit. Each data point represents an average of 5 to 15 segments. The trend lines shown are obtained from Equation 8 for an access density of 40 points/mi.
The trends shown in Figure 3 indicate that free-flow speed is typically larger than the speed limit. However, the relationship between the two speeds is not consistent with the guidance documented previously and shown in Figure 2. This difference is due to the fact that speed limit is correlated with access point density—higher speed limits are associated with lower densities. If a mathematic relationship for this correlation is substituted for the density variable in Equation 8, then the resulting trend obtained from Equation 8 is more nearly like that found for \( HCM \) Chapter 21, as shown in Figure 2.

The trends in Figure 3 suggest that, for a given access point density, the free-flow speed and the speed limit will equal one another. For example, for an access point density of 40 points/mi, the trends suggest that the free-flow speed tends to equal the speed limit on rural arterials when that limit is 65 mph. This equality exists at about 45 mph for urban collector and arterial streets, and about 30 mph for urban local streets. The trends suggest that drivers are content with these three speed limits when traveling on the corresponding road type. However, it also suggests that drivers will likely adopt a free-flow speed that is in excess of the speed limit if that limit is posted at a lower value.

**Influence of Segment Length.** The trends in the running time rates listed in Table 4 suggest that speed is influenced by segment length. These rates can be converted into an estimate of the speed associated with a specific segment length and free-flow speed. The resulting relationships are shown in Figure 4 for the four urban street classes defined in the \( HCM \) procedure. The speed categories listed in the figure represent a specified free-flow speed.

The trends in Figure 4 indicate that, for any given free-flow speed and street class, speed increases with segment length. The \( HCM \) does not explain why speed is lower on shorter segments, nor can an explanation be found in the literature.
Prassas (12, 13) developed a set of tables that relate running speed to free-flow speed, segment length, and arterial traffic volume. The tabulated speeds were based on a series of simulation analyses using CORSIM to model urban streets ranging in length from 590 to 5280 ft. The input free-flow speed equaled 35, 45, and 55 mph. Traffic volumes ranged from 200 to 700 veh/h/ln. A plot of the tabulated running speeds found for free-flow speeds of 35 and 55 mph is shown in Figure 5. Similar trends were observed for a free-flow speed of 45 mph. Also shown in this figure is the trend line from Figure 4 for the stated free-flow speed.

Figure 4. Influence of segment length on cruise speed.

Figure 5. Relationship between segment length, volume, and speed.
The trend lines in Figure 5 indicate a sensitivity to segment length. However, the sensitivity found by Prassas differs from that in the HCM, especially for segment lengths of 0.4 mi or less. The trend lines offered by Prassas also exhibit a sensitivity to lane volume that is not present in the HCM trend lines. A discussion of the effect of volume on speed is provided in the next subsection.

It is rationalized that the effect of segment length found in Prassas’ data is a reflection of platoon dispersion. Specifically, the platoons formed by the upstream signal disperse with increasing travel time along the downstream segment. Thus, the average traffic speed is highest on longer segments as platoons disperse. In contrast, platoon dispersion is negligible on shorter segments and platoon speed is limited to that of the vehicles leading the platoon. This effect of segment length on speed is independent of volume, provided that there are enough vehicles to form a platoon of two or more vehicles during each cycle.

**Effect of Vehicle Proximity**

A relationship between speed and volume has been found in numerous examinations of highway traffic flow. A family of models that describe this relationship is described by May (14). He used the following relation to describe this family, where specific values of $L$ and $m$ can be used to describe various speed-volume-density relationships:

$$S_v = S_f \left[1 - \left(\frac{k}{k_j}\right)^{L-1}\right]^{1/(1-m)} \quad (9)$$

where,

- $S_v =$ running speed based on volume, mph;
- $k =$ density ($= v/S_v$), veh/h/ln;
- $v =$ flow rate, veh/h/ln;
- $k_j =$ jam density ($= 5280/d_s$), veh/mi/ln;
- $d_s =$ distance between vehicles in a stopped queue ($= 25$ ft/veh), ft/veh; and
- $L, m =$ calibration coefficients.

Of particular note is the relationship developed by Greenshields (15). It is obtained when $m = 0$ and $L = 2$.

The structure of Equation 9 indicates that running speed is directly related to free-flow speed. Alternatively, the term enclosed in brackets in Equation 9 can be thought of as an adjustment factor for vehicle proximity. As used in Equation 5, this factor would be computed as:

$$f_v = \left[1 - \left(\frac{k}{k_j}\right)^{L-1}\right]^{-m} \quad (10)$$

The ability of Equations 9 and 10 to explain the relationship between speed, volume, and density on multilane highways is well documented (14). However, data provided by May (14, p. 295) suggests that a similar relationship exists for multilane urban streets. These data are shown in Figure 6 using solid data points and represent measurements on one street. Only the data points associated with uncongested conditions are shown (i.e., those with a density less than 30 veh/mi/ln).
They indicate the street has a free-flow speed of about 27 mph. The running speed is shown to decrease slightly with increasing lane flow rate. Equation 9 provides a reasonably good fit to these data when $S_f = 27$, $m = 0$, and $L = 2$ (i.e., Greenshields’ model).

Figure 6. Running speed as a function of flow rate.

The running speeds tabulated by Prassas (12, 13) also demonstrate a relationship between speed and volume. This relationship is shown in Figure 6 for base free-flow speeds of 35, 45, and 55 mph. The tabulated values are shown as open circles in the figure. The best-fit trend line is also shown for each set of data points. The trend lines indicate that speed is about 6 mph below the free-flow speed when the flow rate is 700 veh/h/ln. This trend is consistent with the data provided by May (14).

**Residual Start-Up Lost Time**

As suggested by Equation 5, segment running time includes a component of residual start-up lost time. The control delay procedure is intended to account for start-up lost time in the control delay estimate. Analysts typically use a start-up lost time of 2.0 s when using this procedure. However, a kinematic analysis indicates that start-up lost time is actually larger than 2.0 s (16).

The running time rates listed in Table 4 were examined to quantify the residual start-up lost time that they may reflect. This time was computed using the following equation:

$$T_s = \frac{L}{5280} \left( t_R - \frac{3600}{S_R} \right)$$

where,

$T_s$ = residual start-up lost time, s;

$t_R$ = running time rate (from Table 4), s/mi; and

$S_R$ = running speed, mph.
The running speed used in Equation 11 was obtained from the running time tables developed by Prassas \((12, 13)\), as discussed previously with respect to Figure 5. Values were interpolated where necessary and reflect a volume level of 400 veh/h/ln. The results of this analysis are shown in Figure 7. Each data point in this figure corresponds to one of the running time rates listed in Table 4.

![Figure 7. Residual start-up lost time as a function of segment length.](image)

The trend in the data in Figure 7 indicates that residual start-up lost time decreases from about 5.0 s to -2.0 s as segment length increases from 0.1 to 1.0 mi. It is rationalized that this trend reflects the tendency for a driver to reach a speed that is higher than the maximum queue discharge speed on longer segments. This higher speed reduces the running time on longer segments by an amount that is larger than the increase in start-up lost time due to the higher speed. It is also likely that the trend reflects the tendency for shorter segments to have a greater proportion of their vehicles stopped at the upstream intersection.

**Delay Due to Turning Vehicles**

As suggested by Equation 5, a third component of running time consists of the delay to through vehicles that follow vehicles turning from the major street. This delay can be incurred at any unsignalized access point along the street. It can be caused by vehicles turning to the left or the right into the access point.

**Delay Due to Left -Turns from the Major Street.** For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane. Chapter 17 of the *HCM* describes a model for estimating this delay. Bonneson and
Fitts (17) and Brown et al. (18) have also examined the effect of left-turning vehicles on through vehicle delay. Both groups of researchers have developed a procedure for quantifying this effect.

The aforementioned procedures were used to quantify the delay due to left-turns from the major street. While the results varied slightly among procedures, there was general agreement that the delay ranged from 0.5 to 1.5 s/veh for typical volume combinations. In general, it may appear that this delay is relatively small, from the standpoint of its contribution to the total delay for a typical urban street segment. However, it could be significant if the street is undivided and has frequent driveways, high through volume, and high left-turn volume.

Delay Due to Right -Turns from the Major Street. For right-turn vehicles, the delay results when the following vehicles’ speed is reduced to accommodate the turning vehicle. Bonneson (19) and Brown et al. (18) have examined the effect of right-turning vehicles on through vehicle delay. Both groups of researchers have developed a procedure for quantifying this effect.

The aforementioned procedures were used to quantify the delay due to right-turns from the major street. While the results varied slightly among procedures, there was general agreement that the delay ranged from 0.2 to 1.0 s/veh for typical volume combinations. In general, it may appear that this delay is relatively small, from the standpoint of its contribution to the total delay for a typical urban street segment. However, it could be significant if the street has frequent driveways, high through volume, and high right-turn volume.

Signal Control Delay

This section examines five factors that have been shown to have an effect on signal delay. These factors are arrival type, arrival flow profile, spillback, upstream volume metering, and coordinated-actuated phase duration. The first two factors are related to the signal coordination aspects of the signalized intersections bounding the subject street segment. Metering relates to the regulation of flow to a downstream intersection by an upstream signalized intersection. Spillback relates to the temporary restriction of discharge from an intersection due to the presence of a downstream queue. Phase duration has a fundamental impact on the delay incurred by the movements served by the phase.

Arrival Type

The progression adjustment factor $PF$ is used in Equation 1 to estimate the delay when arrivals to the intersection are influenced by upstream signals. The magnitude of this adjustment factor is based on the green-to-cycle-length ratio and the estimated arrival type. Values for this factor can be determined using Exhibit 15-5 in the HCM, they range from 1 (excellent progression) to 6 (poor progression).

Qualitative guidance is provided in the HCM to help analysts determine which of six arrival types best describe the pattern of arrivals to the through lane group. However, several researchers have noted that the subjective assessment of arrival type can lead to significant error in delay prediction (20, 21). An analysis of the variability in the delay estimate indicates that the use of
arrival type as a descriptor of arrival pattern increases the uncertainty in the delay estimate. Specifically, the standard deviation of the computed delay ranges from 3 to 6 s/veh due to the uncertainty associated with the arrival type estimate.

**Arrival Flow Profile**

Traffic flowing between signalized intersections can exhibit a wide range of flow rates, depending on the upstream signalization, traffic movement volumes, and segment length. At the upstream intersection, a platoon departs at the start of green with a flow rate equal to the saturation flow rate. As the platoon travels down the street, it tends to disperse such that its flow rate decreases. The arrival flow profile at the downstream intersection typically has a sinusoidal shape due to dispersion. The time relationship between the discharge profile at the upstream intersection and the arrival profile at the downstream intersection is dictated by the signal timing offset. It should be noted that this offset is not an input to the HCM methodology.

The effect of signal offset and platoon dispersion on delay is shown in Figure 8a for two street segments. One segment is 1400 ft in length; the other is 5280 ft in length. The downstream signal on both segments has a green-to-cycle-length ratio of 0.68 and a volume-to-capacity ratio of 0.87. The delay data were obtained using TRANSYT-7F (22). For the range of offsets shown, delay ranges from 4 to 21 s/veh for the 1400 ft segment and 10 to 14 s/veh for the 5280 ft segment. The reduced delay range for the longer segment is due to the effect of platoon dispersion. As segment length increases, platoons disperse and the arrival flow profile becomes more uniform. The resulting delays approach those that would be found at an isolated intersection.

Figure 8a illustrates the effect of offset and platoon dispersion on signal control delay. Figure 8b illustrates the effect of offset and platoon dispersion on platoon ratio. Platoon ratio is directly correlated with arrival type. This relationship is shown on the y-axis in Figure 8b. For the 1400 ft segment, the trends in this figure indicate that arrival type varies from 2 to 4 depending
on the signal offset. In contrast, the 5280 ft segment has an arrival type of 3, regardless of signal offset. This trend indicates that arrivals are effectively random at distances of one mile or more.

Queue Spillback

Spillback can be characterized as one of two types: cyclic and sustained. Cyclic spillback can occur each signal cycle and is a result of queue growth during the red indication. When the green indication is presented, the queue dissipates and spillback is no longer present for the remainder of the cycle. This type of spillback can occur on short street segments with relatively long signal cycle lengths.

Sustained spillback occurs at some point during the analysis period and is a result of oversaturation (i.e., more vehicles discharging from the upstream intersection than can be served at the subject downstream intersection). The queue does not dissipate each cycle. Rather, it remains present until the downstream capacity is increased or the upstream demand is reduced.

Both types of spillback are associated with significant delay. However, many of the delayed vehicles are stored on the approaches to the upstream intersection. As a result, the spillback-related delay incurred on the subject segment is limited by the number of vehicles that can be stored on the segment.

Upstream Metering

Tarko (23) noted that the planning and preliminary design applications, for which the HCM methodology is often used, are based on estimated future volumes. He pointed out that, when forecast volumes are used, an upstream signalized intersection can “meter” the number of vehicles that arrive to the downstream intersection. In this context, the output from the upstream signal is limited by its capacity. If the volume arriving at a downstream signal is metered by upstream signal capacity, it may not equal the volume obtained from a forecast or estimation technique. From a facility standpoint, this metering process implies that an oversaturated arterial is likely to have the most significant queuing at the first signal and, possibly, on the side street approaches.

Tarko (23) used the HCM methodology to evaluate a street segment with and without consideration of the effects of upstream metering. He demonstrated that the travel speed estimate obtained when considering metering could be up to 10 mph faster than that obtained when metering was not considered.

Coordinated-Actuated Phase Duration

The HCM methodology was developed with the expectation that phase interval times will be provided by the analyst. This expectation is reasonable when the signal system is operated in a pretimed mode. However, the green interval duration for an actuated phase is not something that is readily available and the HCM offers limited guidance on its estimation.
Appendix B of *HCM* Chapter 16 describes a framework for estimating phase duration for coordinated semi-actuated operation. However, the discussion is general, and a methodology describing the sequence of calculations is not provided. The discussion that is provided indicates that the following controller settings will need to be provided by the analyst to estimate green interval duration for the coordinated and non-coordinated phases:

- Ring termination mode: simultaneous, independent
- Detector length (by phase)
- Minimum green setting (by phase)
- Maximum green setting (by phase)
- Passage time (by phase)
- Recall mode (by phase)
- Cycle splits/force-off & yield points (by phase)

As this list indicates, the amount of information needed to model coordinated semi-actuated operation is significant. However, if provided, the average green interval duration can likely be estimated with reasonable accuracy.

One setting that is not provided in the aforementioned list is force-off mode. Two modes are commonly used to force-off the non-coordinated phases, they are: “fixed” and “floating.” Floating force-off effectively retains cycle time that is unused by the non-coordinated phases and returns it as a very early return to the coordinated phase. In contrast, fixed force-off points allow unused cycle time to be used by the subsequent phase, regardless of whether it is a coordinated or non-coordinated phase. Most modern signal controllers automatically compute these points based on the phase splits and phase sequence entered by the engineer.

**EVALUATION OF PERFORMANCE PREDICTION PROCEDURES**

This part of the chapter summarizes an evaluation of alternative performance prediction procedures. The focus of this evaluation is on procedures that predict measures that describe the operational performance of automobile traffic flow on urban streets. These measures include running time, delay, and stop rate. The objective is to identify procedures that should be considered for inclusion in the methodology described in Chapter 15 of the *HCM* (1). The findings from this evaluation were used to define a plan for developing and calibrating the needed procedures. These procedures are described in Chapter 3.

The procedures embodied in the *HCM* methodology form the basis for this evaluation. A procedure is defined herein as a set of models that are sequentially used to compute a specific quantity, typically an important traffic characteristic or performance measure. A “model” is defined to be an equation or set of equations that describe the relationship between a set of system state variables (i.e., inputs) and a specified system response variable (i.e., output). A methodology is defined to be a set of procedures that can be used to evaluate a specific system (e.g., an urban street segment). Chapter 15 of the *HCM* describes a “methodology.”
The approach taken in this evaluation is that the HCM methodology represents a reasonable starting point to which some enhancements should be made to improve its system modeling capability. A review of the literature (documented in the previous parts) was used to identify (1) weaknesses of the HCM methodology and (2) alternative analytic methods that could be used to overcome these weaknesses.

The focus of the evaluation is on two procedures in the HCM methodology. One procedure is used to estimate running time and the other is used to estimate signal control delay. The next section summarizes an evaluation of the running time prediction procedure. The subsequent section evaluates the signal control delay prediction procedure. In both parts, the discussion is focused on factors that (1) have been documented as having an influence on urban street traffic operations, and (2) may not be adequately accounted for in the HCM methodology.

**Running Time**

There are several factors that influence the running time incurred by motorists traveling along an urban street. They tend to slow (or delay) motorists by increasing running time beyond that required when traveling at the free-flow speed. The amount of delay associated with these factors tends to be small, when compared to the delay incurred at a signalized intersection. However, it can be significant in some specific situations and should be part of a detailed evaluation of segment running time. These factors include:

1. influence of segment length on free-flow speed,
2. delay due to vehicles turning right from a through lane,
3. delay due to vehicles turning left from a through lane,
4. factors influencing free-flow speed (e.g., access point density, lane width, lateral clearance),
5. delay due to proximity of other vehicles (i.e., effect of traffic density on speed), and
6. delay due to parking maneuvers.

This list ranks the factors in terms of their associated delay influence and the frequency with which they are likely to be encountered on most urban streets (the most important factor is listed first).

**Signal Control Delay**

There are several controller functions and flow processes identified in the literature that have an influence on control delay at a coordinated-actuated intersection. Based on this review, the following list ranks the factors that significantly influence control delay and which could be more accurately modeled in the HCM methodology:

1. basic signal coordination (i.e., platoon dispersion),
2. green interval timing (i.e., average phase duration),
3. semi-actuated signal coordination (i.e., signal offset relationship), and
4. upstream signal metering and queue spillback.
This list ranks the factors in terms of their associated delay influence and the frequency with which they are likely to be encountered on most urban streets (the most important factor is listed first).

**Stop Rate**

Traffic control devices separate vehicles on conflicting paths by requiring one vehicle to stop or yield to the other vehicle. The stop causes delay to be incurred and it also has a cost associated with it in terms of fuel consumption and wear on the vehicle. For this reason, information about stops incurred is useful for performance evaluation and the calculation of road user costs. This measure is typically expressed in terms of “stop rate,” which represents the count of stops divided by the number of vehicles served. Stop rate has units of “stops per vehicle.”

Stops are generally expected by motorists when arriving at an intersection as a minor movement (e.g., a turn movement or a through movement on the minor street). However, through drivers do not expect to stop when traveling along a major street. Their expectation is that the signals will be coordinated to some degree such that they can arrive at each signal in succession while it is displaying a green indication for the through movement. For this reason, stop rate is a useful performance measure for evaluating coordinated signal systems. Unfortunately, the HCM methodology does not currently have a procedure for estimating stop rate.
CHAPTER 3

INTERPRETATION, APPRAISAL, AND APPLICATIONS

This chapter describes an interpretation, appraisal, and application of the procedures developed for this research project. These procedures were developed for the urban streets performance evaluation methodology described in Chapter 15 of the *Highway Capacity Manual (HCM)* (1). Issues and limitations associated with this methodology are identified in Chapter 2. The procedures described in this chapter are intended to enhance the *HCM* methodology by improving the accuracy of its performance predictions. A stop rate prediction procedure was developed to extend the range of performance measures predicted by the *HCM* methodology.

This chapter consists of two parts. The first part describes the proposed urban streets methodology framework. The second section provides a summary of the procedures developed for this framework that enhance or extend the existing procedures. The last section summarizes a verification of the methodology with the proposed procedures.

PROPOSED METHODOLOGY FRAMEWORK

Figure 9 illustrates the proposed framework for the urban streets evaluation methodology. The framework identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the figure. The methodology is shown as it would be applied to one direction of travel along one urban street segment. It should be repeated for the evaluation of the other direction of travel if the street has two-way traffic flow.

The shapes used in the figure indicate the type of activity undertaken at each step of the calculation process. The parallelograms indicate steps in the process where input data are needed. The rectangles with rounded corners indicate steps in the process where a procedure is followed to estimate a key variable used in the evaluation. The diamond shape indicates a decision point.

The framework illustrates the calculation process as applied to two system types: coordinated and non-coordinated. The analysis of coordinated systems recognizes the influence of an upstream signalized intersection on the performance of the street segment. The analysis of non-coordinated systems is based on the assumption that arrivals to each boundary intersection are random. The procedures developed for this research are focused on the evaluation of coordinated signal systems (i.e., the portion of the framework shown on the right of Figure 9). However, some of these procedures can be extended to the analysis of non-coordinated systems (as shown in the portion of the framework on the left of Figure 9).
Figure 9. Proposed urban street methodology framework.

For each of the two system types, the framework is subdivided into the planning and operational analysis levels. The preliminary design analysis level follows that shown for planning. The planning and preliminary design analyses focus on an evaluation of the segment through...
movement and do not require detailed input data. In fact, the planning and preliminary design analyses may use default values to further reduce input data needs. In contrast, an operational analysis considers all intersection traffic movements and related signal timing. As such, detailed inputs are needed to describe the flow characteristics, geometry, and signal timing associated with each movement.

The framework is further subdivided into the type of traffic control used at the intersections that bound the segment. This approach recognizes that a segment boundary can be a signalized intersection, roundabout, or all-way stop-controlled intersection. Although not shown, the boundary intersection could also be an interchange ramp terminal. However, the methodology described in HCM Chapter 26 is more appropriate for the evaluation of segments bounded by signalized interchange ramp terminals.

Examination of the framework indicates that the operational analysis of a coordinated-actuated street system consists of the largest number of computational steps. This level of analytic detail is dictated by the need to accurately model the many traffic flow processes that occur along a coordinated urban street as well the operation of the actuated traffic signal controller. For planning and preliminary design analyses, several of these steps are omitted and default values are substituted for the variables that would otherwise be computed for an operational evaluation. These steps are described in more detail in Appendix H.

Finally, it should be noted in Figure 9 that there is reference to various procedures described in HCM Chapters 16, 17, and a forthcoming chapter addressing roundabout intersections. With regard to Chapter 16, the procedure for estimating actuated phase duration at isolated intersections is needed for the analysis of fully-actuated and semi-actuated intersections on non-coordinated street systems. Also, the procedure for estimating control delay in Chapter 16 is needed for the estimation of segment through movement delay. The delay estimation procedure for roundabouts and all-way stop-controlled intersections is needed from their respective chapters for the analysis of non-coordinated systems.

**PROPOSED PROCEDURES**

The proposed methodology consists of several new or revised procedures. These procedures are summarized in this part of the chapter. Table 6 lists each procedure and identifies the appendix within which a more detailed description of the procedure is provided.

**Delay Due to Turning Vehicles**

Vehicles turning from the major street into an access point can cause a delay to following through vehicles. For right-turn vehicles, the delay results when the following vehicles’ speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane.
TABLE 6 Procedures developed for the proposed methodology

<table>
<thead>
<tr>
<th>Procedure</th>
<th>Appendix with Full Documentation by Performance Measure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Running Time</td>
</tr>
<tr>
<td>Delay due to turning vehicles</td>
<td>Appendix A</td>
</tr>
<tr>
<td>Running time</td>
<td>Appendix B</td>
</tr>
<tr>
<td>Arrival flow profile</td>
<td>--</td>
</tr>
<tr>
<td>Actuated phase duration</td>
<td>--</td>
</tr>
<tr>
<td>Stop rate</td>
<td>--</td>
</tr>
<tr>
<td>Capacity constraints</td>
<td>Appendix F</td>
</tr>
</tbody>
</table>

Note:
“--” - not applicable.

On an undivided street, the magnitude of the delay due to turning vehicles for a typical access point can range from 0.04 to 0.72 s/veh/pt. Although this delay is relatively small, it can accumulate to a significant level if there are many access points along the segment. A procedure for estimating this delay is described in Appendix A.

Running Time

There are two principal components of the time that a vehicle takes to travel the length of a street segment, they are: (1) segment running time, and (2) control delay at the downstream signalized intersection. The factors that contribute to segment running time are the subject of this section. These factors are identified in Equation 12. A procedure for estimating running time is described in Appendix B.

\[
T_R = T_s + T_{ff} f_v + \sum_{i=1}^{N_{ap}} d_{ap,i} + d_{other}
\]  

(12)

where,

- \( T_R \) = segment running time, s;
- \( T_s \) = residual start-up lost time, s;
- \( T_{ff} \) = free-flow running time (= 0.682 \( L / S_f \)), s;
- \( L \) = segment length, ft;
- \( S_f \) = free-flow speed, mph;
- \( f_v \) = proximity adjustment factor;
- \( d_{ap,i} \) = delay due to left or right-turns from the street into access point \( i \), s/veh;
- \( N_{ap} \) = number of access points along the subject segment, approaches;
- \( d_{i} \) = delay due to the proximity of other vehicles, s/veh; and
- \( d_{other} \) = delay due to other sources (e.g., curb parking, pedestrians, bicyclists, etc.), s/veh.

As suggested by Equation 12, segment running time represents the sum of four time components. The first component represents the residual start-up lost time. The control delay procedure is intended to account for start-up lost time in the control delay estimate; however, default values of start-up lost time used for signal evaluation are typically lower than the lost time that is actually incurred (particularly the portion that occurs downstream of the intersection). The residual
The start-up lost time term is intended to compensate for this omission by including the remaining lost time as a component of running time.

The second component of Equation 12 represents the product of the free-flow running time and the proximity adjustment factor. The free-flow running time represents the time required to travel the segment at the free-flow speed. This speed represents the speed chosen by the average driver during low-volume conditions and as he or she may be influenced by the street environment. Elements of the street environment that may influence driver speed choice under free-flow conditions include speed limit, access point density, median type, curb presence, and segment length. Empiric evidence suggests that a “short” segment length tends to influence a driver’s choice of free-flow speed. Shorter segments have been found to have a slower free-flow speed, all other factors being the same.

The proximity adjustment factor adjusts the free-flow running time to account for the effect of traffic density. The adjustment results in an increase in running time (and a corresponding reduction in speed) with an increase in volume. The reduction in speed is a result of shorter headways associated with the higher volume and the driver’s propensity to be more cautious when headways are short.

The third component of Equation 12 represents the delay to through vehicles that follow vehicles turning from the major street. This delay can be incurred at any unsignalized access point along the street. For right-turn vehicles, the delay results when the following vehicles’ speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane.

The fourth component of Equation 12 accounts for the many other factors that could cause a driver to incur delay when traveling along a street. For example, a vehicle that is completing a parallel parking maneuver may cause following vehicles to incur some delay. Also, vehicles that yield to pedestrians at a mid-segment crosswalk may incur delay. Finally, bicyclists riding in the traffic lane or in an adjacent bike lane may directly, or indirectly, cause vehicular traffic to adopt a lower speed and incur additional delay.

**Arrival Flow Profile**

Typically, there are three upstream traffic movements that depart at different times during the signal cycle, they are: minor-street right turn, major-street through, and minor-street left turn. Traffic may also enter the segment at various mid-block access points. Figure 10 illustrates how these movements joined by the arrival flow profile procedure to form the arrival flow profile for the subject downstream intersection.

In application, a platoon dispersion model is used to estimate the arrival flows for each movement at the downstream intersection. These arrival flow profiles are shown in the second x-y
plot in the figure. Although not shown, arrivals from mid-segment access points are assumed to have a uniform arrival flow profile (i.e., a constant flow rate for all time steps).

Finally, an origin-destination matrix is used to distribute each arrival flow profile to each of the downstream exit movements. The four arrival flow profiles associated with the subject exit movement are added together to produce the combined arrival flow profile. This profile is shown in the third x-y plot. The upstream movement contributions to this profile are indicated by arrows.

Figure 10. Arrival flow profile prediction.
Comparison of the profiles in the first and second x-y plots of Figure 10 illustrates the platoon dispersion process. In the first x-y plot, the major street through movement has formed a dense platoon as it departs the upstream intersection. However, when this platoon reaches the downstream intersection it has spread out over time and has a lower peak flow rate. In general, the amount of platoon dispersion increases with increasing segment length. For very long segments, the platoon structure degrades and arrivals become uniform throughout the cycle (i.e., random).

The combined arrival flow profile is used to estimate the proportion of vehicles that arrive during the effective green period of the phase that controls the segment through lane group. Figure 10 illustrates the combined arrival flow profile for the major-street through lane group; however, a similar profile can be constructed for turn-related lane groups if they exist. This proportion is then used with the signal control delay procedure to estimate the delay to the through lane group at the downstream signal. The sequence of calculations associated with arrival flow profile prediction is described in Appendix C.

Coordinated-Actuated Phase Duration

There are two phase types at a coordinated-actuated intersection: coordinated and non-coordinated. The duration of a coordinated phase is dictated by the cycle length and the force-off settings for the non-coordinated phases. These settings define the points in the signal cycle at which each non-coordinated phase must end. The force-off settings are used to ensure that the coordinated phases receive a green indication at a specific time in the cycle. Presumably, this time is synchronized with the coordinated phase time at the adjacent intersections such that traffic is progressed along the street segment. In general, the duration of a coordinated phase is equal to the cycle length less the time allocated to the conflicting phase in the same ring and the time allocated to the minor street phases. Detectors are not typically assigned to the coordinated phase and this phase is not typically extended by the through vehicles it serves.

The duration of a non-coordinated phase is dictated by detected traffic demand in much the same manner as is an actuated phase. However, the non-coordinated phase duration is typically constrained only by its force-off setting (rather than a maximum green setting). The duration of a non-coordinated phase is comprised of four time periods. The first period represents the time lost while the queue reacts to the signal indication changing to green. The second interval represents the time required to clear the queue of vehicles. The third period represents the time the green is extended by randomly arriving vehicles. It ends when there is a gap in traffic (i.e., gap out) or the green extends to the maximum limit (i.e., max out). The last period represents the change period. The duration of a non-coordinated phase can be expressed by the following equation:

\[ D_p = l_1 + g_s + g_e + Y \]  

(13)

where,
\[ D_p = \text{phase duration, s;} \]
\[ l_1 = \text{start-up lost time, s;} \]
\[ g_s = \text{queue service time, s;} \]
\[ g_e = \text{green extension time, s;} \]
and
\[ Y = \text{combined yellow change and red clearance interval duration, s.} \]
The relationship between the variables in Equation 13 is shown in Figure 11 using a queue accumulation polygon. Also shown in this figure is the relationship between the equation variables and the queue length during the average signal cycle. During the red interval, vehicles arrive and form a queue. The queue reaches its maximum length \( l_1 \) seconds after the red interval ends. At this time, the queue begins to discharge at a rate equal to the saturation flow rate \( s \) less the arrival rate during green \( q_g \). The queue clears \( g_s \) seconds after it first begins to discharge. Thereafter, random vehicle arrivals to the intersection are detected and extend the green interval. Eventually, a gap occurs in traffic (or the maximum green limit is reached) and the green interval ends. The end of the green interval coincides with the end of the extension time \( g_e \).

![Figure 11. Time elements influencing non-coordinated phase duration.](image)

A desirable feature of coordinated-actuated signal operation is its ability to adjust the green interval duration to cyclic variation in traffic demand. Although an actuated phase may vary in duration on a cycle by cycle basis, it can be adequately represented by its long-run average duration for the purpose of operational evaluation. Specifically, this average duration can be used to estimate the delay or stops incurred by traffic movements served by the phase. A procedure for estimating the duration of the non-coordinated and coordinated phases is described in Appendix D.

**Stop Rate at a Signalized Intersection**

Stop rate is defined as the average number of full stops per vehicle. A full stop is defined to occur when a vehicle slows to zero (or a crawl speed, if in queue) in response to a change in signal indication from green to red. An arrival-departure polygon is shown in Figure 12 to illustrate the concept of full stop, as opposed to a partial stop. Each solid thin line in the figure that angles upward from left to right represents the trajectory of one vehicle. The time between trajectories represents the headway between vehicles (i.e., the inverse of flow rate \( q \)). The slope of the trajectory represents the vehicle’s speed. The curved portion of a trajectory indicates deceleration or acceleration. The horizontal portion of a trajectory indicates a stopped condition. The effective red \( r \) and effective green \( g \) times are dimensioned at the top of the figure.
Figure 12 shows the trajectories of eight vehicles. The first five trajectories (counting from left to right) have a horizontal component to their trajectory that indicates they have reached a full stop as a result of the red indication. The sixth trajectory has some deceleration and acceleration but the vehicle does not stop. This trajectory indicates a partial stop was incurred for the associated vehicle. The last two trajectories do not incur deceleration or acceleration, and the associated vehicles do not slow or stop. Thus, the number of full stops $N_f$ is 5 and the number of partial stops $N_p$ is 1. The total number of stops $N_t$ is 6. The full stop rate is 0.63 stops/veh (= 5/8). A procedure for estimating stop rate is described in Appendix E.

**Capacity Constraints**

When the volume for an intersection traffic movement exceeds its capacity, the volume discharging from the intersection is restricted (or metered). When this metering occurs for a movement that enters the subject segment, the volume arriving at the downstream signal is reduced below the unrestricted value.

To determine if metering occurs, the capacity of each upstream movement that discharges into the subject segment must be computed and checked with the associated demand volume. If this volume exceeds movement capacity, then the volume entering the segment must be reduced to equal the movement capacity. A procedure for determining if capacity constraints exist and adjusting the upstream volumes is described in Appendix F.

**VERIFICATION**

This part of the chapter describes the activities undertaken to verify the accuracy of the proposed urban streets methodology. The objective of this verification process is to demonstrate the
ability of the methodology to accurately predict automobile performance for a wide range of conditions. The verification was based on a comparison of performance estimates from the engine with those obtained from the CORSIM traffic simulation model. Four street segments were selected for this evaluation. The findings from this evaluation are summarized in this part. Additional discussion of the verification process is provided in Appendix G.

The delay, stop rate, and travel speed for all four sites combined are shown in Figure 13. Also shown is the root mean square error for each of the regression analyses. This variable provides an indication of the standard deviation of the prediction. It is based on one hour of simulation for each data point. Smaller values are likely for longer simulation runs. Figure 13a indicates that the proposed methodology is inclined to yield delays that are 1 to 2 s larger than those predicted by CORSIM. This difference is likely due to the different platoon dispersion models being used, especially in terms of when each model predicts the arrival of the platoon.

![Graphs showing performance measure predictions](image)

- **a. Control Delay - All Sites.**
- **b. Stop Rate - All Sites.**
- **c. CORSIM Travel Speed - All Sites.**
- **d. HCM Travel Speed - All Sites.**

*Figure 13. Comparison of performance measure predictions.*
The stop rate predicted by the methodology is shown in Figure 13b. The quality of fit is similar to that for delay. The methodology is able to explain 88 percent of the variability in the CORSIM predicted stop rate. The standard deviation of the estimate from the methodology is 0.07 stops/veh.

Figure 13c indicates that the proposed methodology is a reasonably good predictor of the CORSIM travel speed, especially for longer segments with higher speed. It has a tendency to estimate slightly slower speeds on shorter segments with lower speeds. The standard deviation of the estimate is 1.5 mph. The methodology explains about 97 percent of the variability in the CORSIM predicted speeds.

Figure 13d compares the travel speed estimated using the methodology in Chapter 15 of the HCM with the travel speed predicted by the proposed methodology. The delay model is common to both the HCM and the proposed methodology so the comparison is based on the running time predicted by the two methodologies (i.e., the delay used in the travel speed estimate is the same for both methodologies). The trend in Figure 13d indicates that the proposed methodology is a good predictor of the HCM predicted travel speed. In contrast to the trend in Figure 13c, the two methodologies are in agreement on travel speed for shorter segments with slower speeds. Also in contrast to the trend in Figure 13c is the tendency of the proposed methodology to estimate slightly higher speeds on longer segments (i.e., higher by about 6 percent). It appears that CORSIM and the HCM have more disagreement with each other about the true travel speed than with the proposed methodology. The speed predicted by the proposed methodology tends to have a value that is between that obtained from CORSIM and the HCM.
CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The research conducted for this project has led to the formulation of several conclusions. These conclusions are described in four sections. Each section addresses one of the following topics: running time, arrival flow profile, coordinated-actuated phase duration, and stop rate.

Running Time

Segment running time represents the sum of four time components. The first component represents the residual start-up lost time. This component is intended to compensate for the omitted start-up lost time that occurs when a start-up lost time value less than 6.0 s is used to estimate control delay.

The second component represents the product of the free-flow running time and the proximity adjustment factor. The free-flow running time is influenced by speed limit, access point density, median type, curb presence, and segment length. The proximity adjustment factor adjusts the free-flow running time to account for the effect of traffic density. The adjustment results in an increase in running time with an increase in volume.

The third component represents the delay to through vehicles that follow vehicles turning from the major street. This delay can be incurred at any unsignalized access point along the street. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane.

The fourth component accounts for the many other factors that could cause a driver to incur delay when traveling along a street (e.g., curb parking, pedestrians, bicyclists).

Arrival Flow Profile

The platoon dispersion model developed by Robertson (24) can be modified to exclude the platoon dispersion calibration coefficient $\alpha$ and the platoon arrival time calibration coefficient $\beta$. The modification allows the model to be calibrated using average travel time as the only input variable.

The reduction in platoon flow rate at increasing distance from the upstream signal is a result of two phenomena: platoon dispersion and platoon decay. Platoon dispersion occurs as platooned vehicles spread out with increased travel time from the upstream signal. The model developed by Robertson (24) is appropriate for modeling platoon dispersion. Platoon decay occurs as vehicles depart from the platoon (or join the back of it) at mid-segment access points. The origin-destination distribution of upstream traffic movements to downstream lane groups is appropriate for modeling platoon structure decay.
A sensitivity analysis indicates that platoon decay tends to have a more significant impact on the arrival flow profile than platoon dispersion. Accurate modeling of platoon dispersion (in the absence of decay effects) requires minimizing the effects of platoon decay by selecting sites with negligible access point activity, or by collecting data flow profile data for only those vehicles that traverse the entire segment. If these types of measures are not taken and the platoon dispersion model is calibrated with traffic streams that include access point traffic, the calibrated smoothing factor will reflect both dispersion and decay effects and its value will be larger than that if it represented only dispersion.

Coordinated-Actuated Phase Duration

Control delay for traffic movements served by a coordinated or non-coordinated signal phase can be accurately estimated using a deterministic delay equation. However, it requires, as an input variable, an estimate of the average phase duration for the analysis period. The calculation of average phase duration should include consideration of the “maximum allowable headway” that characterizes the detection design associated with the phase. It should also include consideration of the force-off and yield point settings for each phase and whether the phase sequence includes leading or lagging left-turn phases.

Stop Rate

The stop rate for traffic movements served by a coordinated or non-coordinated signal phase can be accurately estimated using a deterministic equation that is similar in structure to the deterministic delay equation. The estimated stop rate represents the proportion of vehicles that stop in response to a change in signal indication from green to red. Based on this definition, the stop rate can exceed 1.0 if a vehicle is stopped because of a cycle failure (i.e., a phase terminates because it has reached its maximum green duration and there is one or more queued vehicles left unserved).

RECOMMENDATIONS

The research conducted for this project has led to the formulation of several recommendations. These recommendations are described in the following paragraphs.

It is recommended that the proposed methodology described in Appendix H be considered as a replacement for the methodology in Chapter 15 of the HCM.

Even though the proposed methodology has addressed the most significant limitations of the methodology in Chapter 15 of the HCM, several additional limitations remain that should be the subject of future research. These limitations are summarized in the following paragraphs.

With regard to running time, research is needed to evaluate the effect of on-street parking activity, trucks, and grade on running time. If these factors are found to have some influence, then the research should develop models for quantifying their impact to segment running time.
With regard to actuated controller operation, research is needed to extend the actuated phase duration prediction procedure to account for pedestrian detection and right-turn phase overlap.

With regard to stop rate prediction, research is needed to evaluate the effect of mid-segment turns on through vehicle stop rate. If turns into a mid-segment access point are found to have a significant effect on running time, then this research should develop a model for quantifying this impact.

With regard to capacity constraints, research is needed to develop a procedure for estimating phase capacity when frequent cyclic spillback occurs. This type of spillback occurs when the maximum back of queue from the downstream signal backs into the subject intersection briefly each signal cycle.

Finally, research is needed on the operational effects of added lanes on the approach and departure legs of an intersection (also called “flared” approaches). This research should specifically examine the effects of these lanes on lane utilization and on segment operation.
REFERENCES


APPENDIX A

Procedure for Estimating Delay Due to Turning Vehicles
APPENDIX A

PROCEDURE FOR ESTIMATING DELAY DUE TO TURNING VEHICLES

INTRODUCTION

This appendix describes a procedure for quantifying the delay due to vehicles turning from the major street at an unsignalized access point. This delay is incurred by vehicles that are following the turning vehicles but are not turning at the access point (i.e., they are “through” vehicles). Typically, turn-related delay at an access point is small, relative to that incurred at a signalized intersection. However, this delay can accumulate to notable levels when a street segment has frequent access points and a significant number of turning vehicles.

An access point is defined to be any unsignalized access location along a street. It can be either a driveway or a public street approach. For simplicity, the term “intersection” is used hereafter to describe the junction between a street and an access point. Also, the street at this intersection is referred to as the “major street” and the intersecting roadway is referred to as the “minor street.” The through traffic movement on the major street is referred to as the “priority” movement. All other access-point-related movements are “non-priority” movements. All access points referred to in this appendix are assumed to be “active” in the sense that they have sufficient volume to have some impact on major-street traffic operations. An active access point is defined to have an entering volume of 10 veh/h or more during the analysis period. Through lane location on multilane intersection approaches is characterized using the terms “inside” and “outside.” The outside lane is furthest to the right, when facing the intersection, and the inside lane is furthest to the left.

This appendix consists of four main parts. The first part reviews selected procedures described in the literature that can be used to quantify the delay due to turning vehicles. The second part describes a procedure for computing this delay. The third part describes the findings from a sensitivity analysis using the procedure. The last part summarizes the findings from the calibration of selected models using field data.

BACKGROUND

This part of the appendix describes two procedures for predicting the delay due to vehicles that turn at an access point. The first procedure predicts the delay due to vehicles turning right from the major street. It is described in the first section. The second procedure predicts the delay due to vehicles turning left from the major street. It is described in the second section. The third section describes a model for predicting the flow rate in each through traffic lane on the approach to the access point. Lane flow rate is used in each of the two delay-prediction procedures to determine the number of vehicles delayed by each turning vehicle.
**Delay Due to Right-Turns**

This section describes the model developed by Bonneson (1) for estimating the delay to through vehicles that follow a vehicle turning right from the major street. Through vehicles are delayed when they have to reduce speed because of a right-turning vehicle in the lane.

The model requires as input the flow rate in the outside through lane. For single-lane approaches, this flow rate equals the combined flow rate of the left, through, and right-turn movements. For multi-lane approaches, the outside lane flow rate is dependent on both the amount of right-turning activity in the outside lane and left-turn activity in the inside through lane. A lane flow rate prediction model is described in a subsequent section.

The delay process is modeled using a time-space representation of traffic flow on a street where each vehicle has the same speed $u$, as shown in Figure A-1. The trajectories of the right-turn $R$ and following through vehicles $T$ are sequentially evaluated in the model to determine the average through vehicle delay as a consequence of the turn. The right-turn vehicle trajectory initiates the slowing process in the outside through lane. The following vehicle may have to slow to avoid the right-turn vehicle if it is closely following this vehicle. A second, third, fourth, etc. through vehicle may also have to slow to maintain a minimum headway $h$ between it and the through vehicle ahead. The delay $d$ incurred by each through vehicle is computed as the time lag in its trajectory.

![Figure A-1. Time-space representation of through and right-turn vehicle trajectories.](image)

The event sequence modeled is initiated by the arrival of a right-turn vehicle and ends with the arrival of a subsequent right-turn vehicle. Any through vehicles between these two right-turn vehicles may be delayed, if they are closely following the right-turn vehicle. The sequence has a relatively short duration, no vehicles come to a stop, and the delays incurred by each vehicle are
generally only a few seconds in duration. For these reasons, it is assumed that lane changing by through drivers to avoid a slowing right-turn vehicle is negligible during the event sequence.

A sensitivity analysis was conducted by Bonneson (1) using the model to explore the effect of various factors on through vehicle delay. The factors explored include the outside lane flow rate, right-turn speed, and percentage of right-turns in the outside lane. Flow conditions in the outside lane were assumed to be uncongested, having an average flow rate of 1,000 veh/h/ln or less. This assumption insures that each event is independent of any preceding event. The findings from the analysis are shown in Figure A-2.

Figure A-2. Effect of flow rate, percentage of right turns, and turn speed on through vehicle delay.

The trend lines in Figure A-2 indicate that through vehicle delay increases with lane flow rate. They also indicate that through vehicle delay increases as the portion of right-turn vehicles increases or as the right-turn maneuver speed decreases.

The range of through vehicle delay illustrated in Figure A-2 is lower than that typically experienced by non-priority movements at two-way stop-controlled intersections. Nevertheless, the large number of vehicles that typically incur this delay can yield a large “total” delay (in terms of vehicle-seconds of delay). In fact, in some situations the total delay incurred by the through movement can be larger than that incurred by a non-priority movement.

Through vehicle delay is incurred at each access point. Thus, if a street segment has 10 access points, each of which causes an average 1.0 s/veh delay, the total increase in running time is 10 s/veh.

Bonneson (1) verified model accuracy by comparing estimates from it with data from the TRAF/NETSIM (2) simulation model. TRAF/NETSIM is a stochastic, microscopic simulation.
model that uses car-following logic to move individual vehicles along the simulated street segment and queue-discharge logic at signalized intersection approaches. TRAF/NETSIM reports several performance measures including average delay, travel time, queue length, and percent stopping.

The through delay comparison between TRAF/NETSIM and the model was conducted by establishing a hypothetical street segment with a single access point at the middle of the segment. This segment was 1300 ft length, had two through lanes in each direction, a free-flow speed of 40 mph, and was bounded at each end by a signalized intersection. Flow rates on the major street ranged from 500 to 800 veh/h/ln. The percentage of right turns ranged from 0 to 20 percent. Two sets of simulation runs were conducted. The first set of simulation runs had no right-turning vehicles. These runs were used to quantify the through delay due solely to the density of the traffic stream, as predicted by TRAF/NETSIM.

The second set of simulation runs included right turn at the mid-segment access point. It was used to quantify the delay due to vehicle density and right-turning vehicles. The delays obtained from the first set of runs were subtracted from those obtained from the second set of runs to obtain the delays due to the right-turn activity. The duration of each simulation run was dictated by the desire to measure the delay due to a minimum of 100 right-turn vehicles; however, a minimum of one-hour of simulated time was applied in all cases.

The results of the verification are shown in Figure A-3. Each data point in this figure represents the average through vehicle delay for one simulation run. The line shown in the figure is not a best-fit trend line; rather, it represents the set of all points where the dependent $y$ and independent $x$ variables are equal. Based on the clustering around the “$y = x$” line, Bonneson (1) concluded that the proposed model is able to predict the delays incurred by major-street through drivers with reasonable accuracy.

![Figure A-3. Comparison of delays predicted by the procedure and by simulation.](image-url)
Delay Due to Left-Turns

This section describes the procedure developed by Bonneson and Fitts (3) for estimating the delay to through vehicles that follow a vehicle turning left from the major street. This delay is incurred when the left-turn queue exceeds the available storage and blocks the adjacent through lane (in this context, the undivided cross section is modeled as a major street approach having no left-turn storage). The following through vehicles are delayed when they stop behind the queue of turning vehicles. This delay ends when the left-turn vehicle departs or the through vehicle merges into the adjacent through lane. By merging into the adjacent lane, drivers reduce their delay relative to the delay they would have incurred had they waited for the left-turn queue to clear.

The procedure requires as input the flow rate in the inside through lane. For single-lane approaches, this flow rate equals the combined flow rate of the left, through, and right-turn movements. For multilane approaches, the inside lane flow rate is dependent on the amount of left- and right-turning activity and whether the left-turn related queue blocks the inside through lane. A lane flow rate prediction model is described in the next section.

The delay process is modeled by estimating traffic conditions when the inside through lane on the major street is blocked by a left-turn queue. These conditions include the inside lane flow rate, the delay incurred should the through driver chose to wait for the left-turn queue to clear, and the delay incurred should the through driver chose to merge into the adjacent through lane. The delay incurred during the blocked condition is assumed to be the smaller of the two delay cases. This delay is then multiplied by the probability that the inside lane is blocked to yield the average delay to the through traffic stream.

A sensitivity analysis was conducted by Bonneson and Fitts (3) to explore the effect of various factors on through vehicle delay. The factors explored include the approach flow rate and the percentage of left-turn vehicles. The major street cross section was undivided (i.e., no left-turn storage was available). The findings from the analysis are shown in Figure A-4.

The trends shown in Figure A-4 indicate that the delay to through vehicles on the subject approach tends to be low relative to the delays typically incurred by non-priority traffic movements at unsignalized intersections. This trend is due to a large number of undelayed through vehicles. In general, through vehicle delay increases with increasing approach flow rate.

The effect of left-turn percentage is not consistent in Figure A-4. The trends in the figure indicate that delay increases with left-turn percentage for low to moderate flow rates; however, this trend does not hold for high flow rates. For high flow rates, the highest delays result from the “6 percent” left-turn case. This trend is likely a reflection of through drivers becoming more inclined to avoid the inside through lane as left-turn activity increases beyond 6 percent. It suggests that there may be a left-turn percentage associated with the maximum delay for high flow rates such that left-turn percentages higher or lower would yield lower delay.

Chapter 17 of the Highway Capacity Manual (HCM) (4) describes a procedure for estimating the delay to through vehicles due to left-turning activity. The delay predicted using this procedure
is shown in Figure A-4. The trends in this figure indicate that this procedure overestimates delay, relative to that predicted by the model developed by Bonneson and Fitts (3).

Figure A-4. Effect of flow rate and percentage of left turns on through vehicle delay.

Benneson and Fitts (3) verified the accuracy of their model by comparing it to data from the TWLTL-SIM simulation model. This model was developed by Ballard and McCoy (5) for evaluating the operational effects of alternative mid-segment left-turn treatments. It is a microscopic, stochastic model that can be used to quantify delays and stops to all traffic movements on two, four, and six through lane arterial segments bounded by signalized intersections. The TRAF/NETSIM simulation model was considered for this verification but was found to be very limited in its ability to model driver lane-choice decisions in the vicinity of unsignalized intersections.

Benneson and Fitts (3) compared the through delays predicted by their model with those obtained from TWLTL-SIM using a range of approach flow rates and left-turn percentages. Both two and three-lane approaches were considered in this comparison. The cross section was undivided. The street segment modeled was 1300 ft in length, had two or three through lanes in each direction, a free-flow speed of 40 mph, and was bounded at each end by a signalized intersection. The flow rate on the major street ranged from 500 to 800 veh/h/ln. The percentage of left turns varied from 5 to 20 percent. The duration of each simulation run was dictated by the desire to measure the delay due to a minimum of 100 left-turn vehicles; however, a minimum of one-hour of simulated time was applied in all cases.

The results of the verification are shown in Figure A-5. Each data point in this figure represents the average through vehicle delay for one simulation run. The line shown in the figure represents the set of all points where the dependent and independent variables are equal. In general, the data in this figure suggest that the two models are in general agreement. However, there is a slight tendency for the model to predict lower delays than TWLTL-SIM for the three-lane case.

A-6
Some of the discrepancy is due to limitations of the TWLTL-SIM model. More details on these limitations are provided by Bonneson and McCoy (6). Based on this analysis, Bonneson and Fitts (3) concluded that the model is able to replicate the effect of approach flow rate and left-turn percentage on through vehicle delay.

![Figure A-5. Comparison of delays due to left turns predicted by the model and by simulation.](image)

**Lane Flow Rate**

Bonneson and McCoy (6) developed a lane flow rate model to predict the flow rate in each through lane on the approach to an access point. It uses the flow equilibrium concept to estimate the distribution of traffic to each through lane. This concept is based on the assumption that drivers will minimize their travel time through the intersection or access point. This trait can be mathematically described as a desire to minimize the volume-to-saturation ratio for the subject approach. The minimum ratio is achieved when the volume-to-saturation ratio for each approach lane is equal to that of the other lanes.

The aforementioned concept is similar to that used in Chapter 16 of the *Highway Capacity Manual* (4) for computing the proportion of left-turning vehicles in the inside lane of a shared-lane signalized intersection approach. However, the model developed by Bonneson and McCoy applies to access point intersections and includes a sensitivity to the proportion of right-turn vehicles in the outside lane.

Input variables for the lane prediction model include: the flow rate on the intersection approach, number of through lanes, left-turn percentage, right-turn percentage, left-turn storage length, left- and right-turn equivalency factors, and traffic condition. The storage length variable allows the model to be used with divided and undivided cross sections, where the latter cross section is modeled by assigning it a storage length of 0.0 vehicles. The equivalency factors represent the
ratio of the turn vehicle headway to that of the through vehicle. The value of this factor can vary, depending on turn speed and left-turn capacity. Values for the typical right-turn vehicle range from 1.5 to 2.5. Values for left-turn vehicles can be much larger when the left-turn capacity is small.

The model was derived such that two traffic conditions could be represented. One condition is described as “blocked” and occurs when the inside through lane is occupied by one or more queued left-turn vehicles. When this condition exists, arriving through drivers often attempt to merge into an adjacent through lane. Many through drivers are able to complete this merge without stopping; however, those drivers that stop have to wait until the turn vehicle departs or until there is an opportunity to merge into the adjacent lane. The capacity of the stopped merge maneuver is dependent on the flow rate in the adjacent traffic lane while the left-turn queue is present.

The second condition is described as “unblocked.” It exists whenever the inside through lane is not blocked by a left-turn queue. While this condition exists, lane flow rates tend to be somewhat balanced. However, the number of vehicles in each lane may not be exactly equal because some through drivers will choose to avoid the “threat” of delay due to a possible turning vehicle by choosing a lane where the presence of turning vehicles is negligible.

The model developed by Bonneson and McCoy (6) was subsequently updated by Bonneson and Fitts (3) to reflect consideration of the probability of a lane-change occurring. They noted that the original version of the model overestimated the through lane flow rate in the outside lane under very low and very high flow rate conditions. Under very low flow rate conditions, they rationalized that drivers were not motivated to change lanes because the frequency of left turns was low and the threat of delay was negligible. Under very high flow rate conditions, they rationalized that through drivers did not have an opportunity to move to the outside lane due to the lack of adequate gaps in this lane. Only at moderate flows were drivers both motivated to change lanes and had opportunity to do so. On this premise, they incorporated a sensitivity to the probability of a lane-change in the original model.

A sensitivity analysis was conducted using the updated model to explore the effect of approach flow rate and left-turn percentage on lane volume. Specifically, the model was used to estimate the percentage of vehicles in the inside through lane of a two-lane intersection approach with 6 percent right-turns. The findings from this analysis are shown in Figure A-6.

The trend lines in Figure A-6 indicate that, during unblocked periods, the flow rate in the inside lane is about one-half (i.e., 50 percent) of the approach flow rate. The amount varies from 47 to 53 percent, depending on the left-turn percentage; however, it was insensitive to the approach flow rate. These findings suggest that drivers distribute themselves somewhat evenly among approach lanes when there are no potential disruptions (or delays) due to left-turning vehicles.

During “blocked” periods, the volume in the inside lane varies widely as a function of left-turn percentage and approach volume. Specifically, the volume in inside lane decreases as the left-turn percentage or approach lane volume increases. These findings suggest that through drivers change from the inside lane to the outside lane to avoid the delay associated with left-turn-related queues. More drivers change lanes as the potential delay increases.
DETERMINING DELAY DUE TO TURNS

Vehicles turning from the major street often delay the following through vehicles. This delay can be incurred at any unsignalized access point along the street. For right-turn vehicles, the delay results when the following vehicles’ speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane.

Procedures for estimating these two delays are described in this part of the appendix. These procedures are based on the assumption that the segment traffic flows are random. While this assumption may not be strictly correct for urban streets, it is conservative in that will yield slightly larger estimates of delay. Moreover, expansion of the models to accommodate platooned flows would not likely be cost-effective given the small amount of delay caused by turning vehicles.

Delay Due to Right-Turns

A vehicle turning right from the major street into an access point often delays the through vehicles that follow it. Through vehicles are delayed because they have to reduce speed to avoid a collision with the vehicle ahead, the first of which has reduced speed to avoid a collision with the right-turning vehicle. This delay can be several seconds in duration for the first few through vehicles but will always decrease to negligible values for subsequent vehicles as the need to reduce speed diminishes. For purposes of running time calculation, this delay must be averaged over all through vehicles traveling in the subject direction. The resulting average delay is computed as:

---

Figure A-6. Effect of percentage left turns and approach volume on lane volume distribution.
\[
\text{\textit{d}}_{\text{opr, r}} = 0.67 \left( \text{\textit{d}}_{\text{tr}} \frac{P_{\text{RT}}}{1 - P_{\text{LT}} - P_{\text{RT}}} \right) 
\]
(1)

where,
\(d_{\text{opr}}\) = through vehicle delay due to right turns, s/veh;
\(d_{\text{tr}}\) = through vehicle delay per right-turn maneuver, s/veh;
\(P_{\text{RT}}\) = proportion of right-turn vehicles in the approach traffic stream; and
\(P_{\text{LT}}\) = proportion of left-turn vehicles in the approach traffic stream.

The variable \(d_{\text{tr}}\) in Equation 1 converges to 0.0 as the proportion of turning vehicles approaches 1.0. The constant “0.67” represents a calibration factor based on field data. The steps undertaken to quantify this factor are described in the next part of the appendix. Equation 1 can also be used to estimate the delay due to left-turn vehicles on a one-way street. In this case, variables associated with the right-turn movement would be redefined as applicable to the left-turn movement and vice versa.

As indicated by Equation 2, the delay due to right turns is based on the value of several variables. The following sequence of computations can be used to estimate these values.

1. Compute the minimum speed for the first through vehicle \(u_m\),

\[
u_m = 1.47 S_f - r_d \left( H_1 - \frac{1}{\lambda} \left( \frac{H_1}{H_1 - \lambda} \right) \right) \geq u_r
\]
(2)

with,
\[
\frac{1}{\lambda} = \frac{1}{\frac{1}{\Delta} - \Delta}
\]

(3)

\[
H_1 = \frac{1.47 S_f - u_r}{r_d} + t_c + \frac{L_v}{1.47 S_f} > \Delta
\]
(4)

\[
\lambda = \frac{1}{\frac{1}{\Delta} - \Delta}
\]

(5)

\[
\Delta = \frac{v_a}{3600} \frac{\Delta}{n}
\]
(6)

where,
\(u_m\) = minimum speed of the first through vehicle given that it is delayed, ft/s;
\(u_r\) = right-turn speed, ft/s;
\(h_{\Delta<h<H_1}\) = average headway of those headways between \(\Delta\) and \(H_1\), s/veh;
\(\Delta\) = minimum vehicle headway (use 1.5 s), s/veh;
\(H_1\) = maximum headway that the first through vehicle can have and still incur delay, s/veh;
\(r_d\) = deceleration rate (use 6.7 ft/s\(^2\)), ft/s\(^2\);
\(t_c\) = clearance time of the right-turn vehicle (use 0.6 s), s;
\( L_v = \) average vehicle spacing in a stopped queue (use 25 ft/veh), ft/veh;
\( \lambda = \) flow rate parameter, veh/s;
\( q_n = \) outside lane flow rate, veh/s;
\( v_a = \) approach flow rate on the subject approach, veh/h; and
\( n = \) number of through lanes on the subject intersection approach.

The right-turn speed \( u_r \) used in Equations 2 and 4 is likely to be sensitive to access point design, including the approach profile, throat width, and curb radius. For level profiles and nominal throat widths, the speed can vary from 15 to 25 ft/s for radii varying from 20 to 60 ft, respectively. A turn speed of 20 ft/s can be used when information is not available to make a more accurate estimate.

2. Compute the delay to the first through vehicle, \( d_1 \).

\[
d_1 = \frac{(1.47 S_f - u_a)^2}{2 (1.47 S_f)} \left( \frac{1}{r_a} + \frac{1}{r_a} \right)
\]

where,
\( d_1 = \) conditional delay to first through vehicle, s and
\( r_a = \) acceleration rate (say, 3.5 ft/s\(^2\)), ft/s\(^2\).

3. Compute the delay to the second through vehicle, \( d_2 \).

\[
d_2 = d_1 - (h_{H_2-\Delta} - \Delta)
\]

with,
\[
h_{H_2-\Delta} = \frac{1}{\lambda} + \frac{\Delta - H_2 e^{-\lambda(H_2-\Delta)}}{1 - e^{-\lambda(H_2-\Delta)}}
\]

\[
H_2 = d_1 + \Delta
\]

where,
\( d_2 = \) conditional delay to vehicle 2, s.

4. Compute the delay to the third and subsequent through vehicles, \( d_i \) (\( i = 3, 4, \ldots \)).

\[
d_i = d_{i-1} - (h_{H_i-\Delta} - \Delta)
\]

with,
\[
h_{H_i-\Delta} = \frac{1}{\lambda} + \frac{\Delta - H_i e^{-\lambda(H_i-\Delta)}}{1 - e^{-\lambda(H_i-\Delta)}}
\]

\[
H_i = d_{i-1} + \Delta
\]
where,

\[ d_i = \text{conditional delay to the } i\text{th vehicle } (i = 2, 3, 4, \ldots), \text{ s.} \]

As Equations 8 and 11 suggest, the delay to each subsequent through vehicle is less than or equal to that of the preceding vehicle. In fact, the sequence of delays always converges to zero when the average flow rate in the outside lane is less than \(1/\Delta\).

Step 4 should be repeated for the third and subsequent through vehicles until the delay computed for vehicle \(i\) is less than 0.1 s. In general, this criterion results in delay being computed for only the first two or three vehicles.

5. Compute the proportion of right-turning vehicles in the outside through lane \(P_R\).

\[ P_R = P_{R\%} \alpha \leq 1.0 \tag{14} \]

where,

\( P_R = \text{proportion of right-turning vehicles in the outside through lane.} \)

Equation 14 represents an approximation that yields reasonably accurate values for typical right-turn percentages. An equation that can be used to obtain a more accurate estimate of \(P_R\) for a wide range of turn percentages is described by Bonneson and McCoy (6).

6. Compute the through vehicle delay per right-turn maneuver, \(d_{vtr}\). The through vehicle delay for the first two vehicles is computed using the following equation:

\[ d_{vtr} = d_1 \left(1 - e^{-\lambda(H_1 - \Delta)}\right) (1 - P_R) + d_2 \left(1 - e^{-\lambda(H_2 - \Delta)}\right) (1 - e^{-\lambda(B_2 - \Delta)}) (1 - P_R)^2 \tag{15} \]

If three or more vehicles are delayed, then an additional term needs to be added to Equation 15 for each subsequent vehicle. In this situation, the following equation applies to all delayed vehicles:

\[ d_{vtr} = \sum_{i=1}^{n} \left[ d_i \times \prod_{j=1}^{i} \left(1 - e^{-\lambda(H_j - \Delta)}\right) \times (1 - P_R)^j \right] \tag{16} \]

7. Compute the through vehicle delay due to right turns \(d_{vtr}\) using Equation 1.

**Delay Due to Left-Turns**

Through vehicles on the major-street approach to an unsignalized intersection can incur delay when the left-turn queue exceeds the available storage and blocks the adjacent through lane (in this context, the undivided cross section is considered a major street approach having no left-turn storage). The through vehicles that follow are delayed when they stop behind the queue of turning vehicles. This delay ends when the left-turn vehicle departs or the through vehicle merges into the adjacent through lane. By merging into the adjacent lane, drivers reduce their delay relative to the delay they would have incurred had they waited for the left-turn queue to clear. This delay is computed as:
where,

\[ d_{ap,l} = P_{ov} d_{k1} \left( \frac{1}{P_L} - 1 \right) \frac{P_{LT}}{1 - P_{LT} - P_{KT}} \]  \hspace{1cm} (17)

where,

\( d_{ap,l} \) = through vehicle delay due to left turns, s/veh;

\( P_{ov} \) = probability of left-turn bay overflow (i.e., a queue in the inside through lane);

\( d_{k1} \) = average delay to through vehicles in lane 1 (i.e., the inside lane), s/veh; and

\( P_L \) = proportion of left-turning vehicles in the inside through lane.

As indicated by Equation 17, the delay due to left turns is based on the value of several variables. The following sequence of computations can be used to estimate these values.

1. Compute the probability of a lane change, \( P_{lc} \).

\[ P_{lc} = 1 - \left( 2 \frac{v_a}{n s_{lc}} - 1 \right)^2 \]  \hspace{1cm} (18)

where,

\( P_{lc} \) = probability of a lane change;

\( s_{lc} \) = maximum flow rate that will allow any lane changes (= 3,600/\( h_{lc} \)); veh/h/ln; and

\( h_{lc} \) = minimum acceptable headway for a lane change (use 3.7 s), s.

The approach flow rate is equal to the sum of the left-turn, through, and right-turn flow rates on the major-street approach to the subject unsignalized intersection. If the ratio \( \frac{v_a}{(n s_{lc})} \) in Equation 18 exceeds 1.0, then it should be set to 1.0.

2. Compute the through vehicle equivalent for a left-turn vehicle, \( E_L \).

\[ E_L = \frac{s_i}{c_i} \]  \hspace{1cm} (19)

with,

\[ c_i = v_c \frac{e^{-v_c t_c/3600}}{1 - e^{-v_c t_f/3600}} \]  \hspace{1cm} (20)

where,

\( E_L \) = through vehicle equivalent for a left-turn vehicle;

\( s_i \) = saturation flow rate for a through stream (use 1800 veh/h/ln);

\( c_i \) = left-turn capacity, veh/h/ln;

\( v_c \) = conflicting flow rate, veh/h;

\( t_c \) = critical gap for left-turn maneuver (use 4.1 s), s; and

\( t_f \) = follow-up gap for left-turn maneuver (use 2.2 s), s.

Equation 20 estimates the capacity of the left-turn movement. The capacity estimation model provided in Chapter 17 of the HCM can be substituted, if desired.
3. Compute the modified through vehicle equivalency factors, $E_L^*$ and $E_R^*$.

\[ E_L^* = (E_L - 1)P_{lc} + 1 \]  \hspace{1cm} (21)

\[ E_R^* = (E_R - 1)P_{lc} + 1 \]  \hspace{1cm} (22)

where,

$E_L^*$ = modified through vehicle equivalent for a left-turn vehicle;

$E_R^*$ = modified through vehicle equivalent for a right-turn vehicle;

$E_R$ = through vehicle equivalent for a right-turn vehicle (use 2.2); and

$P_{lc}$ = probability of a lane change.

4. Compute the proportion of left-turning vehicles in the inside through lane $P_L$.

\[ P_L = \frac{-b + \sqrt{b^2 - 4Rc}}{2R} \leq 1.0 \]  \hspace{1cm} (23)

with,

\[ b = R - P_{LT}[1 + (n - 1)(2E_L^* - 1)] \]  \hspace{1cm} (24)

\[ c = -P_{LT}n \]  \hspace{1cm} (25)

\[ R = 1 + P_{RT}(E_R^* - 1) \]  \hspace{1cm} (26)

where,

$R, b, c$ = intermediate calculation variables.

If the number of through lanes on the subject intersection approach $n$ is equal to 1, then $P_L = P_{LT}$.

5. Compute the proportion of right-turning vehicles in the outside through lane $P_R$.

\[ P_R = P_{RT}\frac{\frac{1}{s_i} + n - 1}{1 - P_{RT}\left(\frac{1}{s_i} + n - 2\right)(E_R^* - 1)} \leq 1.0 \]  \hspace{1cm} (27)

with,

\[ s_1 = \frac{s_i(1 + P_L)}{1 + P_L(E_L^* - 1) + (P_LE_L^*)} \]  \hspace{1cm} (28)

where,

$s_i$ = saturation flow rate for the inside lane, veh/h/ln.
If the number of through lanes on the subject intersection approach \( n \) is equal to 1.0, then \( P_R = P_{RT} \). If the right-turn vehicles are provided an exclusive right-turn lane, then \( P_{RT} \) should equal 0.0 in Equation 27 and in all subsequent equations.

6. Compute the flow rate in the inside lane \( v_i \) and the outside lane \( v_n \).

\[
v_i = v_a \frac{P_{RT}}{P_R} \quad (29)
\]

\[
v_n = \begin{cases} 
  v_a \frac{P_{RT}}{P_R} & \text{if } P_R > 0.0 \\
  \frac{v_a - v_i}{n - 1} & \text{if } P_R = 0.0 
\end{cases} \quad (30)
\]

where,
- \( v_i = \) flow rate for the inside lane, veh/h and
- \( v_n = \) flow rate for the inside lane, veh/h.

7. Compute the flow rate in the intermediate lanes \( v_2, v_3, \ldots, v_{n-1} \). If there are more than two lanes on the subject intersection approach, then the following equation can be used to estimate the flow rate in the intermediate lanes.

\[
v_j = \frac{v_a - v_1 - v_n}{n - 2} \quad (31)
\]

where,
- \( v_i = \) flow rate for lane \( i \), veh/h (\( i = 2, 3, \ldots, n - 1 \)).

The flow rates in lanes \( 2, 3, \ldots, n - 1 \) are identical and equal to the value obtained from Equation 31.

8. Compute the merge capacity \( c_m \) available to through drivers waiting in the inside lane of a multilane approach.

\[
c_m = v_2 \frac{e^{-v_2 t_m/3600}}{1 - e^{-v_2 t_m/3600}} \quad (32)
\]

where,
- \( c_m = \) merge capacity, veh/h;
- \( v_2 = \) flow rate in the adjacent through lane (i.e., lane 2), veh/h; and
- \( t_m = \) minimum acceptable gap for a merge from a stopped condition (use 3.7 s), s.

The flow rate in the adjacent lane equals \( v_n \) if the approach only has two through lanes.

9. Compute the delay to through vehicles that merge \( d_m \).

\[
d_m = 3600 \left( \frac{1}{c_m} - \frac{1}{v_2} \right) + 900 T \left[ \frac{v_m}{c_m} - 1 + \left( \frac{v_m}{c_m} - 1 \right)^2 + \frac{8 v_m^2}{c_m^2 T} \right] \quad (33)
\]
with,

\[ \nu_m = \nu_1 - P_{LT} \nu_a \geq 0.0 \]  \hspace{1cm} (34)

where,

\[ d_m = \text{merge delay, s/veh}; \]
\[ \nu_m = \text{merge flow rate (i.e., the through flow rate in the inside lane), veh/h; and} \]
\[ T = \text{analysis time period, h.} \]

This delay is incurred by through vehicles that stop in the inside lane and eventually merge in the adjacent through lane. The \(1/s_t\) term included in Equation 33 extracts the service time for the through vehicle from the delay estimate, such that the delay estimate represents the increase in travel time resulting from the left-turn queue.

10. Compute the capacity of the inside lane \(c_{nm}\) for vehicles that do not merge.

\[ c_{nm} = \frac{s_t(1 + P_L)}{1 + P_L(E_L - 1) + (P_L E_L)} \]  \hspace{1cm} (35)

where,

\[ c_{nm} = \text{non-merge capacity, veh/h.} \]

The unadjusted through vehicle equivalent for a left-turn vehicle \(E_L\) is used in this equation to estimate the non-merge capacity.

11. Compute the delay to through vehicles that do not merge \(d_{nm}\).

\[ d_{nm} = 3600 \left( \frac{1}{c_{nm}} - \frac{1}{s_t} \right) + 900T \left[ \frac{\nu_1}{c_{nm}} - 1 + \left( \frac{\nu_1}{c_{nm}} - 1 \right)^2 + \frac{8\nu_1}{c_{nm}^2 T} \right] \]  \hspace{1cm} (36)

where,

\[ d_{nm} = \text{non-merge delay, s/veh.} \]

This delay is incurred by through vehicles that stop in the inside lane and wait for the queue to clear. These vehicles do not merge into the adjacent lane. The \(1/s_t\) term is included in Equation 36 for the same reason as provided in the discussion of Equation 33.

12. Compute the delay to through vehicles in the inside lane \(d_{,1}\). This delay is estimated as the smaller of the delay relating to each maneuver. It is computed as:

\[ d_{,1} = \text{smaller of: } [d_{as}, d_{m}] \]  \hspace{1cm} (37)

13. Compute the probability of left-turn bay overflow \(P_{ov}\).

\[ P_{ov} = \left( \frac{\nu_0 P_{LT}}{c_t} \right)^{N_{\nu,1}} \]  \hspace{1cm} (38)

A-16
where,
\[ N_v = \text{number of left-turn vehicles that can store before spilling back into the inside through lane, veh.} \]

For an undivided cross section, the number of left-turn vehicles that can store \( N_v \) is equal to 0.0.

14. Compute the through vehicle delay due to left-turns \( d_{ap,l} \) using Equation 17.

**SENSITIVITY ANALYSIS**

This part of the appendix examines the models for estimating delay to through vehicles as a result of vehicles turning left or right at unsignalized access points along a major street. The examination focuses on the sensitivity of the delay estimate to variation in approach flow rate and the number of access points.

The analysis is based on several assumptions. First, it was assumed that the major street has an undivided cross section with two through lanes in each direction. The free-flow speed on the segment was assumed to be 45 mph. The segment was 1300 ft in length and the number of access points on each side of the street was 2, 6, or 9. These access point frequencies correspond to densities of about 30, 60, and 90 access points per mile. It was assumed that 20 percent of the traffic entering the segment at its upstream end would exit the major street by turning left at an access point. A similar percentage would exit by turning right at an access point. At each access point, the number of vehicles that exited the major street was matched by an equal number of vehicles that entered the major street by a right or left turn. In this manner, there was no net loss or gain of vehicles along the segment length.

The turn percentage at each access point varied, depending on the number of access points present. For the scenario where there were only two access points along the segment, there were 10 percent left turns and 10 percent right turns at each access point. For the scenario with six access points, there were 3.3 percent left turns and 3.3 percent right turns at each access point. For the scenario with 9 access points, the turn percentage was 2.2 percent. In this manner, the sum of the left-turn percentages for each scenario added to 20 percent for the segment (the same held true for the right-turn percentage).

The models described in the previous part of the appendix were used to compute the through vehicle delay at each access point. These delays were multiplied by the number of access points to estimate the total through vehicle delay along the segment. The findings from this analysis are shown in Figure A-7.

The trend lines in Figure A-7 indicate that the delay to through vehicles increases with increasing approach flow rate and turn percentage. The trends suggest that the amount of delay incurred is less than 2.0 s/veh, and relatively insensitive to the number of access points, when the flow rate is less than 1400 veh/h.
This part of the appendix describes the findings from a calibration of the two models for predicting through vehicle delay as a result of turning vehicles. One model predicts the delay due to right-turning vehicles and the other predicts the delay due to left-turning vehicles. Both models are described in a previous part of this appendix. The first section describes the study sites and summarizes the data collected at each site. The second section summarizes the findings from the model calibration.

Study Site Description and Database Summary

Calibration data were collected at four study sites. Each site represents an intersection on an urban arterial street. Data for calibrating the right-turn-related delay model were collected at two sites. Data for calibrating the left-turn-related delay model were collected at two other sites. Each site was selected because it was observed to have a relatively high volume of turning activity. The two sites for the left-turn-related study are located on undivided streets. Table A-1 summarizes the characteristics of the four study sites.

### Table A-1 Study segment summary table

<table>
<thead>
<tr>
<th>Study Type</th>
<th>Site</th>
<th>Corridor</th>
<th>Intersection</th>
<th>Location</th>
<th>Street Class</th>
<th>Segment Length, ft</th>
<th>Speed Limit, mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Right turn</td>
<td>A</td>
<td>Texas Avenue</td>
<td>Francis Drive</td>
<td>College Station, Texas</td>
<td>II</td>
<td>2610</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>Texas Avenue</td>
<td>Lincoln Street</td>
<td>College Station, Texas</td>
<td>II</td>
<td>2300</td>
<td>40</td>
</tr>
<tr>
<td>Left turn</td>
<td>C</td>
<td>Addison Road</td>
<td>Addison Tower</td>
<td>Addison, Texas</td>
<td>III</td>
<td>2640</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>Koenig Lane</td>
<td>Arroyo Seca</td>
<td>Austin, Texas</td>
<td>III</td>
<td>2980</td>
<td>35</td>
</tr>
</tbody>
</table>
Data were collected for two to three hours at each of the four study sites. Some of the data were collected using tapeswitch sensors adhered to the road surface in advance of the turn location. These sensors were positioned to form a “speed trap” to facilitate the accurate measurement of each vehicle’s speed in advance of the turn location. Some of the data were collected using videotape recorders located in the vicinity of the turn location. Data extracted from the videotape included the time each turn occurred and the time each through vehicle needed to travel through the study segment. This time was used to estimate the increase in travel time (i.e., delay) due to each right-turn maneuver. The statistics in Table A-2 provide an overview of the calibration database.

### TABLE A-2 Data reduction statistics for access point study sites

<table>
<thead>
<tr>
<th>Study Type</th>
<th>Site</th>
<th>Intersection</th>
<th>Study Duration, hrs</th>
<th>Average Speed, mph</th>
<th>Turning Vehicles</th>
<th>Delayed Through Vehicles</th>
<th>Total Vehicles in Subject Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Right turn</td>
<td>A Francis Drive</td>
<td>2.2</td>
<td>38.4</td>
<td>102</td>
<td>205</td>
<td>929</td>
<td></td>
</tr>
<tr>
<td></td>
<td>B Lincoln Street</td>
<td>2.3</td>
<td>32.1</td>
<td>176</td>
<td>311</td>
<td>1314</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total:</td>
<td>4.5</td>
<td></td>
<td>278</td>
<td>516</td>
<td>2243</td>
<td></td>
</tr>
<tr>
<td>Left turn</td>
<td>C Addison Tower</td>
<td>2.6</td>
<td>39.4</td>
<td>112</td>
<td>369</td>
<td>690</td>
<td></td>
</tr>
<tr>
<td></td>
<td>D Arroyo Seca</td>
<td>3.2</td>
<td>38.9</td>
<td>167</td>
<td>606</td>
<td>1341</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total:</td>
<td>5.8</td>
<td></td>
<td>279</td>
<td>975</td>
<td>2031</td>
<td></td>
</tr>
<tr>
<td>Grand Total:</td>
<td></td>
<td>10.3</td>
<td></td>
<td>557</td>
<td>1491</td>
<td>4274</td>
<td></td>
</tr>
</tbody>
</table>

All total, 10 hours of videotape were reviewed. Data for 4274 vehicles were extracted from the videotape using manual methods. All of these vehicles traveled in the “subject” lane, which is the outside lane for the right-turn study and the inside lane for the left-turn study. For the right-turn study, 2243 vehicles traveled in the outside lane. Of these vehicles, 516 (23 percent) were delayed by 278 (12 percent) right-turning vehicles. For the left-turn study, 2031 vehicles traveled in the inside lane. Of these vehicles, 975 (48 percent) were delayed by 279 (14 percent) left-turning vehicles. The delay for each of the delayed through vehicles was estimated from the videotape and tapeswitch data and subsequently entered into a database.

One objective of the study design for the left-turn-related sites was to gather data for through vehicles that chose to merge into the adjacent lane rather than continue to wait in the inside lane for the left-turn queue to clear. However, of the 975 through vehicles that were delayed, only 12 were observed to complete this merge maneuver. Hence, calibration of the left-turn-related model focused on its ability to estimate the delay to the through vehicles that queued in the inside lane.

### Model Calibration

The formulation of the right-turn-related model enabled it to be used to estimate the delay to each through vehicle that followed a right-turn vehicle. Thus, a delay estimate was obtained for each through vehicle in the outside lane. In contrast, the formulation of the left-turn-related delay model is such that it does not provide an estimate of individual vehicle delay. Rather, it yields an
average delay to all through vehicles for the analysis period, as would be obtained by observing all through vehicles for this time period and averaging the delay over all observed vehicles.

As suggested by Table A-2, a total of 516 observations of through vehicle delay were quantified in the right-turn database. For the left-turn database, vehicle counts for five-minute intervals were extracted from the videotape records. Through vehicle delays were measured for each vehicle for the same intervals and averaged to obtain one, five-minute observation. Thus, the right-turn-related database consisted of 516 single-vehicle observations and the left-turn-related database consisted of 70 five-minute observations.

The calibration process consisted of making small adjustments to the model parameters (e.g., deceleration rate, acceleration rate, critical gap, saturation flow rate, etc.) in an iterative manner with the objective of minimizing the sum of the squared error for all observations. The error was computed as the difference between the observed and measured delay. Once the best fit was obtained in this manner, the observed and predicted delays were compared using the following equation:

\[ d_o = b_0 + b_1 d_p \]  \hspace{1cm} (39)

where,
- \( d_o \) = observed delay, s/veh;
- \( b_i \) = calibration coefficient \( i (i = 0, 1) \); and
- \( d_p \) = predicted delay, s/veh.

Least-squares linear regression was used to estimate the calibration coefficients in Equation 39. If \( b_0 \) was found to be significantly different from 0.0, or \( b_1 \) was significantly different from 1.0, then this finding was inferred to be evidence of a bias in the predicted delay estimate. To offset this bias, the coefficient would be incorporated in the delay prediction model.

The relationship between the predicted and observed right-turn-related delays is shown in Figure A-8. Only 51 data points are shown in the figure. Each data point represents the average of ten delay observations. The data points were computed by sorting the predicted delays in ascending order, forming groups of 10 observations, and computing the average delay for each group. These averages were computed to facilitate the graphic portrayal of trend in Figure A-8. It was needed because the presentation of the base 516 data points resulted in a figure that was so “busy” that it was difficult to discern a trend in the data.

The best-fit trend line to the 51 observations is shown in Figure A-8. A regression analysis of the base 516 data points yielded an identical trend line; however, the coefficient of determination \( R^2 \) was reduced to 0.19. The coefficient of 0.67 indicates that the prediction model overestimates the observed delay. The \( b_0 \) coefficient was not significantly different from 0.0. Based on this finding, an empiric adjustment of 0.67 was included in Equation 1.

The relationship between the predicted and observed left-turn-related delays is shown in Figure A-9. Each data point in the figure represents the delay for one five-minute interval. The trend in the data suggested that the predicted delay slightly underestimated the observed delay. The best-
fit trend line is also shown in the figure. It suggests that the predicted delay should be multiplied by 1.05 if it is to be used as an accurate estimate of delay. However, this coefficient is not significantly different from 1.0. Similarly, the $b_0$ coefficient is not significantly different from 0.0. Based on this finding, it was determined that an empiric adjustment was not needed for Equation 17.

![Figure A-8](image1.png)

*Figure A-8. Comparison of predicted and observed delay due to right-turn vehicles.*

![Figure A-9](image2.png)

*Figure A-9. Comparison of predicted and observed delay due to left-turn vehicles.*
REFERENCES


APPENDIX B

Procedure for Estimating Running Time
APPENDIX B

PROCEDURE FOR ESTIMATING RUNNING TIME

INTRODUCTION

This appendix describes a procedure for estimating the running time along an urban street segment. It includes models for estimating the free-flow speed and the influence of traffic density on speed. This increase in density is coincident with an increase in flow rate along the street. It results in a reduction in speed, relative to the free-flow speed, as drivers adapt to the closer proximity of adjacent vehicles. The reduction in speed corresponds to an increase in running time.

This appendix consists of four main parts. The first part describes selected models that can be used to quantify free-flow speed and the delay due to the proximity of other vehicles. The second part describes a procedure for determining segment running time. It identifies the components of running time and highlights the contribution of free-flow speed and delay due to the proximity of other vehicles. The third part describes the findings from a sensitivity analysis using the procedure. The last part summarizes the findings from the calibration of selected models using field data.

BACKGROUND

This part of the appendix describes three models. Initially, a model for estimating free-flow speed is described. Then, a model for estimating the influence of traffic density and flow rate on running speed is described. Finally, a model for estimating residual start-up lost time is described.

Free-Flow Speed Model

This section describes a model for estimating free-flow speed. Free-flow speed represents the average speed of through automobile drivers when traveling along a street under low-volume conditions and when not delayed by any control device or other vehicle. It reflects the effect of the street environment on driver speed choice. It is derived from two models; one model is described by Prassas (1, 2) and the other is described by Fitzpatrick (3).

The model developed by Prassas (1, 2) estimates the effect of segment length on running speed. It was calibrated using data from the CORSIM simulation model (4). The simulation runs reflected a range of inputs, including volume, base free-flow speed, segment length, number-of-lanes, and turn percentage. All runs were based on “excellent” progression such that signal timing allowed the through-vehicle platoon to arrive soon after the start of the green indication. The model was presented in tabular form and included sensitivities to running speed, segment length, and number-of-lanes. As segment length decreased below about 0.4 mi, speed decreased significantly. The effect of number-of-lanes was negligible and, as noted by Prassas, not statistically significant. It should be noted that Prassas also examined the effect of flow rate on running speed. The findings from this examination are described in the next section.
The effect of segment length on free-flow speed, as found in Prassas’ model, is shown in Figure B-1. The running speeds associated with a volume of 200 veh/h/ln were rationalized to be representative of free-flow conditions. These speeds are shown in Figure B-1 for base free-flow speeds of 35, 45, and 55 mph. The data points shown represent the values listed in the tables provided by Prassas. They correspond to segment lengths of 590, 750, 1050, 1760, and 5280 ft.

The thick trend lines in Figure B-1 suggest that a segment length greater than about 2000 ft (0.4 mi) has a negligible effect on free-flow speed. This finding is consistent with that of Fitzpatrick et al. (3) whose data included only segments with a length of 0.4 mi or more.

It is rationalized that the effect of segment length shown in Figure B-1 is a reflection of platoon dispersion. Specifically, the platoons formed by the upstream signal disperse with increasing travel time along the downstream segment. Thus, the average traffic speed increases on longer segments as platoons disperse. In contrast, platoon dispersion is negligible on shorter segments and platoon speed is limited to that of the vehicles leading the platoon. This effect of segment length on speed is independent of volume, provided that there are enough vehicles to form a platoon of two or more vehicles during each cycle.

Also shown in Figure B-1 is the relationship between free-flow speed and segment length implied by Exhibit 15-3 of HCM Chapter 15 (5). The sloped portion of the dashed line implies that the HCM relationship is more sensitive to segment length than suggested by Prassas’ model.

The model developed by Fitzpatrick et al. (3) was based on field measurements of free-flow speed on 35 urban and suburban arterial street segments in six states. The database collectively has segments ranging in length from 0.4 to 1.0 mi; access point densities ranging from 0 to 103 mi, speed limits ranging from 30 to 55 mph; lane widths ranging from 9.5 to 14.0 ft, and median types that
included no median, non-restrictive, and restrictive. Regression analysis was used to calibrate the model. The effects of segment length, access point density, lane width, and median type were not statistically significant and, therefore, not included in the model.

The model reported by Fitzpatrick et al. (3) was based on 85th percentile spot speeds. The final report also provided average free-flow speeds. These speeds were converted into space-mean speeds (using a method described in a later section) and the model was re-calibrated. The results are shown in Figure B-2. The best-fit equation is shown in the figure using a thick trend line. The model structure is the same as that used by Fitzpatrick et al.; however, the coefficient values are slightly different. The coefficient of determination $R^2$ for the model is 0.66, which implies that the model explains 66 percent of the variability in the data.

![Figure B-2. Base free-flow speed as a function of speed limit.](image)

The relationships shown in Figures B-1 and B-2 were used to develop a combined free-flow speed model. It is represented by the following equations:

\[
S_f = S_{fo} f_L \tag{1}
\]

with,

\[
S_{fo} = 18 + 0.61 S_{pl} \tag{2}
\]

\[
f_L = 1.02 - 4.7 \frac{S_{fo} - 19.5}{5280 L} \tag{3}
\]

where,

- $S_f$ = free-flow speed, mph;
- $S_{fo}$ = base free-flow speed, mph;
- $S_{pl}$ = posted speed limit, mph;
\[ f_i = \text{segment length adjustment factor}; \text{ and} \]
\[ L = \text{segment length, mi.} \]

It should be noted that the speeds predicted by Equations 1 and 2 represent average values, as opposed to 85th percentile values. Equation 3 was derived to fit the tabular values reported by Prassas (2) for volumes of 200 veh/h/ln.

### Speed-Flow Model

This section describes the model developed by Prassas (1, 2) for estimating running speed as a function of flow rate. Details of the model development process used by Prassas are summarized in the previous section. The model was presented by Prassas in tabular form and included a sensitivity to segment length and to flow rate. The effect of segment length on free-flow speed was the subject of the previous section. The effect of flow rate on running speed is the subject of this section.

A speed-flow equation was derived to provide a best-fit to the running speed values listed in Prassas’ tabular model. The form selected for the equation is based on the linear speed-density relationship developed by Greenshields (6). The linear speed-density relationship was combined with the fundamental relationship between flow \( v \), speed \( s \), and density \( k \) (i.e., \( v = s k \)) to yield the following speed-flow equation:

\[
S_v = S_f \frac{1}{2} \left( 1 + \left[ 1 - \frac{4v}{S_f k} \right]^{0.5} \right)
\]

where,
- \( S_v = \text{running speed based on volume, mph;} \)
- \( v = \text{mid-segment flow rate, veh/h/ln;} \)
- \( k_j = \text{jam density (= 5280} / d_j, \text{ veh/mi/ln; and} \)
- \( d_s = \text{distance between vehicles in a stopped queue (= 25 ft/veh), ft/veh.} \)

It should be noted that the speed predicted by Equation 4 represents an average value, as opposed to an 85th percentile value. Moreover, it does not include the effect of turbulence in the traffic stream due to sources other than vehicle proximity.

The predictive ability of the speed-flow equation is demonstrated through a comparison between it and the running speeds listed in the table reported by Prassas (2). This comparison is shown in Figure B-3 for base free-flow speeds of 35, 45, and 55 mph. The tabulated values are shown as data points in the figure. The equation predictions are shown as thick trend lines. For common flow rates, the predictions appear to be within 1.0 mph of the values from Prassas’ model. The constants in Equation 4 can be adjusted to more closely approximate the trends shown by the data points. Specification of the appropriate values for these constants is discussed in a later section that addresses model calibration.
Residual Start-Up Lost Time Model

Segment running time can include a component of start-up lost time if this time is not fully accounted for in the control delay estimate. Analysts typically use a start-up lost time of 2.0 s when computing control delay. However, a kinematic analysis indicates that start-up lost time is actually larger than 2.0 s. Equation 5 illustrates the factors that influence start-up lost time (7).

\[ l_1 = T_{pr} + \frac{V_{max}}{A_{max}} \]  

where,
- \( l_1 \) = start-up lost time, s;
- \( T_{pr} \) = additional response time of the first queued driver (=1.03 s), s;
- \( V_{max} \) = maximum queue discharge speed (= 49 ft/s), ft/s; and
- \( A_{max} \) = maximum acceleration (= 6.63 ft/s\(^2\)), ft/s\(^2\).

The values shown in parentheses in the variable list were suggested by Bonneson (7) as being representative and correspond to a start-up lost time of 8.4 s. However, field measurements of flow rate at the intersection stop line indicated the need for an empirical adjustment to the second term in Equation 5. Specifically, these data indicated that the second term should be multiplied by 0.357. This adjustment yields a start-up lost time of 3.7 s.

The running time rates listed in HCM Exhibit 15-3 were examined to quantify the residual start-up lost time that they may reflect. This time was computed using the following equation:

\[ T_s = \frac{L}{5280} \left( t_R - \frac{3600}{S_R} \right) \]  

where,
The running speed used in Equation 6 was obtained from the running time tables developed by Prassas (1, 2). Values were interpolated where necessary and reflect a volume level of 400 veh/h/ln. The results of this analysis are shown in Figure B-4. Each data point in this figure corresponds to one of the running time rates listed in HCM Exhibit 15-3.

![Figure B-4](image)

*Figure B-4. Residual start-up lost time as a function of segment length.*

The trend in the data in Figure B-4 indicates that residual start-up lost time decreases from about 5.0 s to -2.0 s as segment length increases from 0.1 to 1.0 mi. It is rationalized that this trend reflects the tendency for a driver to reach a speed that is higher than the maximum queue discharge speed on longer segments. This higher speed reduces the running time on longer segments by an amount that is larger than the increase in start-up lost time due to the higher speed. It is also likely that the trend reflects the tendency for shorter segments to have a greater proportion of their vehicles stopped at the upstream intersection.

The data in Figure B-4 also indicate that segments characterized as urban street class I tend to have larger residual start-up lost time than segments having other class designations. The running time rates for class I segments were added to the HCM for the 1997 update, whereas those for the other classes were originally introduced in the 1985 edition. It is possible that this difference in development time period may explain the class-related difference shown in Figure B-4.

A best-fit trend line to the data for street classes II, III, and IV is also shown in Figure B-4. The equation associated with this trend line is also shown. The numerator of the equation represents the excess start-up lost time that is not accounted for in the delay calculation. The value of “6.0” in

\[
T_s = \text{residual start-up lost time, s};
\]

\[
t_r = \text{running time rate (from Table 4), s/mi}; \text{ and}
\]

\[
S_r = \text{running speed, mph}.
\]
the numerator represents the mid-point between the 3.7 and 8.4 s start-up lost time range computed in a previous paragraph. The variable $l_1$ represents the start-up lost time used in the delay prediction procedure; it was assumed to equal 2.0 s for this analysis. The constant “400” in the equation represents the regression coefficient.

**DETERMINING RUNNING TIME**

There are two principal components of the time that a vehicle takes to travel the length of a street segment, they are: (1) segment running time, and (2) control delay at the downstream signalized intersection. The factors that contribute to segment running time are the subject of this appendix. They are identified in the following equation:

$$T_R = T_s + T_{ff} f_v + \sum_{i=1}^{N_{ap}} d_{ap,i} + d_{other}$$

where,

- $T_R$ = segment running time, s;
- $T_s$ = residual start-up lost time, s;
- $T_{ff}$ = free-flow running time ($= 0.682 \frac{L}{S_f}$), s;
- $L$ = segment length, ft;
- $S_f$ = free-flow speed, mph;
- $f_v$ = proximity adjustment factor;
- $d_{ap,i}$ = delay due to left or right-turns from the street into access point $i$, s/veh;
- $N_{ap}$ = number of access points along the subject segment, approaches;
- $d_v$ = delay due to the proximity of other vehicles, s/veh; and
- $d_{other}$ = delay due to other sources (e.g., curb parking, pedestrians, bicyclists, etc.), s/veh.

As suggested by Equation 7, segment running time represents the sum of four time components. The first component represents the residual start-up lost time. The control delay procedure is intended to account for start-up lost time in the control delay estimate; however, default values of start-up lost time used for signal evaluation are typically lower than the lost time that is actually incurred (particularly the portion that occurs downstream of the intersection). The residual start-up lost time term is intended to compensate for this omission by including the remaining lost time as a component of running time.

The second component of Equation 7 represents the product of the free-flow running time and the proximity adjustment factor. The free-flow running time represents the time required to travel the segment at the free-flow speed. This speed represents the speed chosen by the average driver during low-volume conditions and as he or she may be influenced by the street environment. Elements of the street environment that may influence driver speed choice under free-flow conditions include speed limit, access point density, median type, curb presence, and segment length. Empiric evidence suggests that a “short” segment length tends to influence a driver’s choice of free-flow speed. Shorter segments have been found to have a slower free-flow speed, all other factors being the same.
The proximity adjustment factor adjusts the free-flow running time to account for the effect of traffic density. The adjustment results in an increase in running time (and a corresponding reduction in speed) with an increase in volume. The reduction in speed is a result of shorter headways associated with the higher volume and the driver’s propensity to be more cautious when headways are short.

The third component of Equation 7 represents the delay to through vehicles that follow vehicles turning from the major street. This delay can be incurred at any unsignalized access point along the street. For right-turn vehicles, the delay results when the following vehicles’ speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane. A procedure for estimating these two delays is described in Appendix A.

The fourth component of Equation 7 accounts for the many other factors that could cause a driver to incur delay when traveling along a street. For example, a vehicle that is completing a parallel parking maneuver may cause following vehicles to incur some delay. Also, vehicles that yield to pedestrians at a mid-segment crosswalk may incur delay. Finally, bicyclists riding in the traffic lane or in an adjacent bike lane may directly, or indirectly, cause vehicular traffic to adopt a lower speed and incur additional delay.

**Free-Flow Speed**

Free-flow speed is defined to be the average speed of through automobile drivers when traveling along a street under low-volume conditions and when not delayed by any control device or other vehicle. It can be measured in the field. The “base” free-flow speed is defined to be the free-flow speed on long street segments. It can also be measured in the field, provided that the segment is about 0.4 mi or more in length. The following sequence of computations can be used to estimate free-flow speed for those situations where it cannot be measured in the field.

1. Compute the base free flow speed $S_{fo}$.

$$S_{fo} = 25.6 + 0.47S_{pl} + 0.015P_{rm} - 0.078\frac{D_{a}}{N} - 0.47I_{curb} - 0.037I_{curb}P_{rm}$$  \hspace{1cm} (8)

where,
- $S_{fo}$ = base free-flow speed, mph;
- $S_{pl}$ = posted speed limit, mph;
- $P_{rm}$ = percent of segment length with restrictive median, percent;
- $D_{a}$ = access point density (total access points on both sides of street), points/mi;
- $N$ = number of through traffic lanes in subject direction of travel; and
- $I_{curb}$ = indicator variable for presence of curb (= 1.0 if curb is present on the right-hand side of the traveled way, 0.0 otherwise).

B-8
Equation 8 is applicable to urban arterial or collector streets with a posted speed limit ranging from 30 to 55 mph. It can be extended to speed limits of 25 and 60 mph, provided that the resulting speed estimate is considered reasonable for the subject street. Equation 8 is not applicable to local streets or rural highways. The constants in Equation 8 represent calibration coefficients based on field data. The steps undertaken to quantify these factors are described in the next part of the appendix.

The speed limit variable is included in Equation 8 as a general descriptor of the street environment and its geometric design. In this sense, speed limit is highly correlated with the environmental and geometric factors that have a direct influence on driver speed choice. As such, it represents a single input variable that is highly correlated with free-flow speed and is used as a convenient way to limit the need for numerous environmental and geometric input data.

The convenience of using speed limit as an input variable comes with a caution—the analyst must not infer a cause-and-effect relationship between the input speed limit and the estimated segment performance measures. More specifically, the computed change in performance resulting from a change in the input speed limit is not likely to be indicative of performance changes that will actually be realized. Research consistently indicates that a change in speed limit has very little effect on the actual operating speed (8, 9, 10, 11).

The speed limit on the segments used to calibrate Equation 8 was noted to be consistent with that found on other streets in its vicinity and consistent with the 85th percentile speed. This consistency should be confirmed for the subject segment when using Equation 8.

The percent of segment length with restrictive median \( P_{rm} \) represents the length of median divided by the adjusted segment length, expressed as a percentage. The length of median does not include the segment length allocated to median openings. The adjusted segment length equals the segment length minus the width of the upstream signalized intersection. For example, consider a 1320-ft segment that has a 50-ft wide upstream intersection and a raised-curb median for its entire length, except for one 40-ft median opening located mid-segment. The value of \( P_{rm} \) for this segment is 97 percent (\( = \frac{[1320 - 50 - 40]}{[1320 - 50]} \times 100 \)). A restrictive median is any median that is not designed to be crossed by vehicular traffic (e.g., depressed median, raised-curb median, etc.).

Access point density for the subject street segment represents the total number of access points (driveways plus public street approaches) along both sides of the street divided by the adjusted length of the segment. The “adjusted length” equals the length of the segment less the width of the upstream intersection. Access point density is divided by the number of through lanes on in the subject segment because empirical evidence indicates that the effect diminishes with increasing number of lanes.

The indicator variable is used to account for the presence of curb on base free-flow speed. Curb presence is based on consideration of the right-hand side of the traveled way. Curb presence on the left-hand side is specified through median type. If a curb is within two or three feet of the traveled way, then the variable should equal 1.0. It should equal 0.0 if a shoulder is present or if a curb is present but it is four feet or more from the edge of traveled way.
The curb and median presence variables (i.e., $I_{\text{curb}}$ and $P_{\text{rm}}$, respectively) interact such that when a raised-curb median is present for the full length of the street and there is a curb on the right-hand side of the street, the net effect is a speed reduction of 2.7 mph. On the other hand, when a depressed median is present and there is no curb, the net effect is a speed increase of 1.5 mph.

2. Compute the segment length adjustment factor $f_L$.

$$f_L = 1.02 - 4.7 \frac{S_f}{5280} - 19.5 \frac{S_f}{L} \leq 1.0$$  \hspace{1cm} (9)

where,

$f_L$ = segment length adjustment factor; and

$L$ = segment length, mi.

If Equation 9 is used to evaluate segments shorter than 0.08 mi, then the adjustment factor should be computed using $L$ equal to 0.08 mi. If the factor obtained from Equation 9 exceeds 1.0, then it should be set equal to 1.0.

3. Compute the free-flow speed $S_f$.

$$S_f = S_{f0} f_L$$  \hspace{1cm} (10)

where,

$S_f$ = free-flow speed, mph.

Adjustment for Vehicle Proximity

An increase in flow rate has been found to be associated with a decrease in the average running speed of the traffic stream. A reduction in speed due to volume translates into longer running time for motorists. This effect is captured in the vehicle proximity adjustment factor used in Equation 7. It is computed using the following equation:

$$f_v = \frac{2}{1 + \left(1 - \frac{\nu}{52.8 S_f}\right)^{0.21}}$$  \hspace{1cm} (11)

where,

$\nu$ = mid-segment flow rate, veh/h/ln.

The constant “0.21” in Equation 11 represents a calibration coefficient based on field data. The steps undertaken to quantify this factor are described in the next part of the appendix.

Residual Start-Up Lost Time

The residual start-up lost time term accounts for the time required to accelerate to the running speed, less the start-up lost time used to compute the through movement delay. It is computed using the following equation:
\[ T_s = 400 \frac{6.0 - l_1}{5280 L} \]  

(12)

where,

\[ l_1 = \text{start-up lost time, s.} \]

The constant “400” in Equation 12 represents a calibration coefficient based on field data. The steps undertaken to quantify this factor are described in the next part of the appendix.

The start-up lost time variable in Equation 12 is set equal to the value used to compute control delay for the movements entering the subject segment at the upstream signalized intersection. The divisor in Equation 12 is an empirical adjustment that minimizes the contribution of this equation for longer segments. It reflects a tendency for drivers to offset this additional time by adopting slightly higher mid-segment speeds. This term in Equation 7 is only applicable when the upstream boundary intersection is signalized.

SENSITIVITY ANALYSIS

This part of the appendix examines the sensitivity of two models described in the previous part. The examination of the free-flow speed model considers the effect of speed limit, median type, access point density, curb presence, and segment length on free-flow speed. The examination of the delay model considers the effect of speed limit, segment length, and flow rate on delay due to the proximity of other vehicles. The sensitivity of the residual start-up lost time model to segment length was shown previously in Figure B-4.

The free-flow speed model was used to predict the free-flow speed of an urban street segment. Various speed limits and segment lengths were considered. The findings from the analysis are shown in Figure B-5.

![Figure B-5](image_url)

- **a. Non-restrictive or no median.**

- **b. Restrictive median.**

*Figure B-5. Effect of speed limit and segment length on free-flow speed.*
The trend lines in Figure B-5 indicate that the free-flow speed increases with speed limit and segment length. The trend lines indicate that the free-flow speed exceeds the speed limit when the speed limit is less than 40 to 45 mph, the exact value dependant on segment length and median type.

The proximity model was used to examine the proximity adjustment factor as a function of flow rates. Various speed limits, segment lengths, and flow rates were considered. The findings from the analysis are shown in Figure B-6 for segments with a non-restrictive median. The factor increases about 10 percent on curbed segments with a restrictive median (e.g., raised-curb median). It decreases about 10 percent on uncurbed segments with a restrictive median (e.g., depressed median).

![Figure B-6. Effect of flow rate and speed limit on the proximity adjustment factor value.](image)

The trend lines in Figure B-6 indicate that the proximity adjustment factor and running time increase with increasing flow rate. They also indicate an increase in factor value with a decrease in speed limit. For a segment length of 0.50 mi, the trends shown correspond to an increase in running time of about 2.0 s when the flow rate is 800 veh/h/ln.

**SELECTED MODEL CALIBRATION**

This part of the appendix summarizes the findings from the calibration of the models described in the previous part. The findings from this activity are summarized in two sections. The first section describes the study sites and summarizes the data collected at each site. The second section summarizes the findings from the model calibration.
Study Site Description and Database Summary

Calibration data for the free-flow speed and speed-flow models were collected at ten urban study sites. Each site represents one direction of travel on one arterial street segment. The sites were selected such that they collectively represented a range of geographic locations, urban street classes, segment lengths, speed limits, and traffic flow rates. Table B-1 summarizes the characteristics of the ten study sites. These sites are referred to herein as the “partial segment” study sites because the data collection activities focused on traffic flow along the middle one-third of each segment.

**TABLE B-1  Partial segment summary table**

<table>
<thead>
<tr>
<th>Site</th>
<th>Corridor</th>
<th>Location</th>
<th>Street Class</th>
<th>Segment Length, ft</th>
<th>Speed Limit, mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Campbell Road</td>
<td>Richardson, Texas</td>
<td>II</td>
<td>1102</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>H. Mitchell Parkway</td>
<td>College Station, Texas</td>
<td>II</td>
<td>1506</td>
<td>45</td>
</tr>
<tr>
<td>3</td>
<td>H. Mitchell Parkway</td>
<td>College Station, Texas</td>
<td>I</td>
<td>2330</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>Valley View Lane</td>
<td>Farmers Branch, Texas</td>
<td>III</td>
<td>2611</td>
<td>35</td>
</tr>
<tr>
<td>5</td>
<td>Wellborn Road</td>
<td>College Station, Texas</td>
<td>II</td>
<td>3704</td>
<td>45</td>
</tr>
<tr>
<td>6</td>
<td>S.R. 823 (Flamingo Rd.)</td>
<td>Fort Lauderdale, Florida</td>
<td>I</td>
<td>2650</td>
<td>50</td>
</tr>
<tr>
<td>7</td>
<td>S.R. 842 (Broward Blvd.)</td>
<td>Fort Lauderdale, Florida</td>
<td>III</td>
<td>996</td>
<td>35</td>
</tr>
<tr>
<td>8</td>
<td>S.E. Powell Boulevard</td>
<td>Portland, Oregon</td>
<td>II</td>
<td>1405</td>
<td>35</td>
</tr>
<tr>
<td>9</td>
<td>S.E. McLoughlin Blvd.</td>
<td>Portland, Oregon</td>
<td>I</td>
<td>2123</td>
<td>45</td>
</tr>
<tr>
<td>10</td>
<td>S.W. Barbur Boulevard</td>
<td>Portland, Oregon</td>
<td>II</td>
<td>2937</td>
<td>35</td>
</tr>
</tbody>
</table>

Note:
1 - Street class designations are based on the description provided in Exhibits 10-3 and 10-4 of the *Highway Capacity Manual* (5).

Table B-2 shows the sites at which field data were collected for the purpose of calibrating the residual start-up lost time model. The sites listed in Table B-2 were specifically selected to offer a range of urban street class, length, and speed limit conditions. These sites are referred to herein as “full segment” study sites because the data collection activities focused on traffic flow along the full length of the segment, including the boundary intersections. Three sites in Oregon are common to Tables B-1 and B-2.

**TABLE B-2  Full segment summary table**

<table>
<thead>
<tr>
<th>Field Study Technique</th>
<th>Site</th>
<th>Segment</th>
<th>Location</th>
<th>Street Class</th>
<th>Segment Length, ft</th>
<th>Speed Limit, mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full Segment</td>
<td>1</td>
<td>Aviation Parkway</td>
<td>Tucson, Arizona</td>
<td>I</td>
<td>2800</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>S.E. McLoughlin Blvd.</td>
<td>Portland, Oregon</td>
<td>I</td>
<td>2123</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>S.W. Barbur Boulevard</td>
<td>Portland, Oregon</td>
<td>II</td>
<td>2937</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>S.E. Powell Boulevard</td>
<td>Portland, Oregon</td>
<td>II</td>
<td>1405</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>E. Speedway Boulevard</td>
<td>Tucson, Arizona</td>
<td>III</td>
<td>950</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Pratt Street</td>
<td>Baltimore, Maryland</td>
<td>IV</td>
<td>410</td>
<td>35</td>
</tr>
</tbody>
</table>
Site Description

The study of each site included a field survey and a videotape recording of traffic for a period of three hours. The site survey was completed prior to the start of each study. It focused on creating a record of the physical features of the street. The following data were recorded during each study: grade, access point type and location, lane width, median type and length, presence of on-street parking, curb presence, and presence of crosswalks. These data are summarized in Table B-3. The reference marks identified in the top portion of the table are described in the next subsection.

<table>
<thead>
<tr>
<th>TABLE B-3  Survey data for partial segment study sites</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial Segment Characteristic</td>
</tr>
<tr>
<td>--------------------------------</td>
</tr>
<tr>
<td>Distance to reference marks, ft</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Partial segment length^2, ft</td>
</tr>
<tr>
<td>Access points^3</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Access point density, points/mi</td>
</tr>
<tr>
<td>Median length^4, ft</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Percent restrictive median, %</td>
</tr>
<tr>
<td>No. of through lanes (both ways)</td>
</tr>
<tr>
<td>Average through lane width, ft</td>
</tr>
<tr>
<td>Grade, % (+ uphill)</td>
</tr>
<tr>
<td>On-street parking length^5, ft</td>
</tr>
<tr>
<td>Curb within 4.0 ft^6</td>
</tr>
<tr>
<td>Horizontal clearance^7, ft</td>
</tr>
<tr>
<td>Marked pedestrian crossings</td>
</tr>
</tbody>
</table>

Notes:
1 - Site numbers correspond to the sites listed in Table B-1.
2 - Partial segment length is the distance between the upstream stop line and the most distant reference mark.
3 - Number of access points on the partial segment. Includes all access points on the right-hand side of the road (in the direction of travel) and those on the left side that are accessible across the median.
4 - Length of partial segment with each median type. Median openings are allocated to the “no median” category.
5 - Length of partial segment with on-street parking on the right-hand side of the road (in the direction of travel).
6 - “Yes” indicates that curb is present within 4.0 ft of the outside edge of traveled way.
7 - Distance between right edge of outside through traffic lane and the face of curb (or nearest vertical obstruction).
As indicated in Table B-3, a range of geometric conditions is represented in the collective set of study sites. The partial segment lengths (as measured from the upstream stop line to the video reference mark furthest downstream) ranged from 610 to 3104 ft. The access point density varied from 0 to 76 access points per mile among the sites. The proportion of the segment length with a restrictive median ranged from 0 to 92 percent.

Selected characteristics of the six full segment study sites are listed in Table B-4. Collectively, the sites exhibit a range in grade, access point density, median type, and number of through lanes. Only S.E. Powell and S.W. Barbur have parallel parking. However, this parking is permitted only for about one-third of the length of the street.

<table>
<thead>
<tr>
<th>Site</th>
<th>Street Segment</th>
<th>Boundary Intersections</th>
<th>Grade¹, percent</th>
<th>Travel Directions</th>
<th>Access Points</th>
<th>Median Type</th>
<th>Through Lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Aviation Parkway</td>
<td>E. 34th Street S. Country Club Road</td>
<td>0.0</td>
<td>2-way</td>
<td>0</td>
<td>Restrictive</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>S.E. McLoughlin Boulevard</td>
<td>S.E. 17th Avenue S.E. Holgate Boulevard</td>
<td>+2.5</td>
<td>2-way</td>
<td>1</td>
<td>Restrictive</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>S.W. Barbur Boulevard</td>
<td>S.W. 19th Avenue S.W. Bertha Boulevard</td>
<td>-3.4</td>
<td>2-way</td>
<td>19</td>
<td>varies</td>
<td>4</td>
</tr>
<tr>
<td>4</td>
<td>S.E. Powell Boulevard</td>
<td>S.E. 21st Avenue S.E. 26th Avenue</td>
<td>+1.0</td>
<td>2-way</td>
<td>9</td>
<td>Non-restrictive</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>E. Speedway Boulevard</td>
<td>N. 4th Avenue N. 6th Avenue</td>
<td>0.0</td>
<td>2-way</td>
<td>5</td>
<td>Non-restrictive</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>Pratt Street</td>
<td>Light Street (St. Paul St.) Calvert Street</td>
<td>0.0</td>
<td>1-way</td>
<td>0</td>
<td>None</td>
<td>6</td>
</tr>
</tbody>
</table>

Note:
1 - Positive grade is uphill. Grade indicated is relative to the travel direction that was the subject of the data collection.

Database Summary

Partial Segment Study. A partial segment field study technique was used to measure traffic events for the portion of the street segment that is not influenced by the acceleration, deceleration, or queuing associated with the signalized intersections that bound the subject street segment. The boundaries of the partial segment are shown in Figure B-7.

As Figure B-7 indicates, the partial segment starts at a distance of about 400 ft downstream of the upstream signalized intersection. It ends at a distance of about 600 ft upstream of the downstream signalized intersection. At some sites, the segment end location was moved further upstream to avoid having the running time measurements influenced by vehicles that were decelerating to join the queue at the downstream intersection.

The videotapes recorded during each partial segment study were replayed for the purpose of manually extracting the data needed for model calibration. Of the three hours of video tape recorded...
at each site, 45 minutes of data were extracted. Specifically, one 15-minute sample of data was extracted from each one-hour videotape for a total of 30, 15-minute samples from the 10 sites.

Figure B-7. Partial segment boundaries and reference mark locations.

The data reduction statistics are listed in Table B-5. As indicated in the last row of this table, a total of 7892 vehicles were tracked along the partial segments during the thirty, 15-minute time periods. Of these, 7424 (94 percent) traveled completely through the partial segment. The remaining vehicles either entered or exited at a mid-segment access point. The database includes a mix of vehicles that are, and are not, considered free-flow vehicles.

The data were aggregated into cycle observations by computing the flow rate and running speed for all through vehicles during one signal cycle, as defined by the upstream signal. A total of 240 cycle observations were included in the database. The number of cycle observations per site ranged from 16 to 31 and reflected a range in cycle lengths used at the sites. To ensure statistical stability in the individual cycle observations, each cycle was required to include a minimum of 10 vehicles for which an average speed could be computed. This criterion eliminated 40 cycle observations, leaving 200 cycle observations for statistical analysis.

Three characteristics were measured for each tracked vehicle: running time, headway, and classification. Running time was computed as the time that lapsed between the vehicle’s arrival to the upstream reference mark and its departure from the most distant downstream reference mark. Vehicle headway was measured at all reference marks. No vehicles were screened from the database during headway calculation to ensure the accuracy of the headway estimate. A vehicle was classified as a “heavy vehicle” if it was one of the following types: single-unit truck, vehicle with a trailer, bus, vehicle with three or more axles.

Running speed and flow rate were computed for each cycle observation using all vehicle observations in the cycle. Running speed is defined as a space mean speed and was computed as the average running time divided into the length of the partial segment. Flow rate was computed as the reciprocal of the average headway. The average headway for one cycle was computed using the headway measurements for each vehicle at each reference mark during the cycle. Thus, if 20 vehicles traveled down a street with two reference marks during one cycle, the average headway was based on 40 headway observations (= 2 × 20).
**TABLE B-5 Data reduction statistics for partial segment study sites**

<table>
<thead>
<tr>
<th>Site</th>
<th>Corridor</th>
<th>From</th>
<th>To</th>
<th>Vehicles Entering¹</th>
<th>Through Vehicles²</th>
<th>Signal Cycles</th>
<th>Cycle Length ³, s</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Campbell Road N. Collins Boulevard</td>
<td>Canyon Creek Drive</td>
<td>1075</td>
<td>1046</td>
<td>31</td>
<td>90, 89, 129</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Harvey Mitchell Parkway Longmire Drive</td>
<td>Southwood Drive</td>
<td>622</td>
<td>622</td>
<td>23</td>
<td>114, 130, 130</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Harvey Mitchell Parkway Welsh Avenue</td>
<td>Rio Grande Boulevard</td>
<td>537</td>
<td>476</td>
<td>26</td>
<td>111, 117, 127</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Valley View Lane Josey Lane</td>
<td>Webb Chapel Road</td>
<td>515</td>
<td>421</td>
<td>31</td>
<td>89, 89, 121</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Wellborn Road George Bush Drive</td>
<td>Holleman Drive</td>
<td>684</td>
<td>600</td>
<td>19</td>
<td>155, 176, 176</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>S.R. 823 (Flamingo Road) S.W. Taft Street</td>
<td>S.W. Johnson St.</td>
<td>1222</td>
<td>1217</td>
<td>19</td>
<td>163, 163, 163</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>S.R. 842 (Broward Boulevard) N.E. 3rd Avenue</td>
<td>Andrews Avenue</td>
<td>613</td>
<td>604</td>
<td>16</td>
<td>158, 162, 161</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>S.E. Powell Boulevard S.E. 21st Avenue</td>
<td>S.E. 26th Avenue</td>
<td>1191</td>
<td>1146</td>
<td>26</td>
<td>85, 120, 120</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>S.E. McLoughlin Boulevard S.E. 17th Avenue</td>
<td>S.E. Holgate Boulevard</td>
<td>933</td>
<td>931</td>
<td>21</td>
<td>135, 152, 152</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>S.W. Barbur Boulevard S.W. 19th Avenue</td>
<td>S.W. Bertha Boulevard</td>
<td>500</td>
<td>361</td>
<td>28</td>
<td>99, 97, 119</td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Total:</strong></td>
<td></td>
<td></td>
<td><strong>7892</strong></td>
<td><strong>7424</strong></td>
<td><strong>240</strong></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
1 - Vehicles entering the site at the upstream intersection. Total for three, 15-minute periods.
2 - Vehicles that entered the segment via a left-turn, right-turn, or through movement at the upstream signal and exited the segment via a left-turn, right-turn, or through movement at the downstream signal.
3 - Each cycle length listed corresponds to one of the three 15-minute analysis periods.

**Full Segment Study.** Four hours of data were collected at each of the six study sites. The data were collected using a combination of videotape recorders, traffic signal status monitoring equipment, and field survey. The camcorders were strategically located along the subject segment such that a view of each bounding signalized intersection was obtained. To the extent possible, the camcorders also provided a view of most of the length of the subject street segment.

Several types of data were collected, they include: traffic, geometry, signal timing, and performance data. The traffic, geometry, and signal timing data were needed to form the input data files for the traffic simulation model. These data included intersection turn movement counts, heavy-vehicle percentage, segment length, intersection geometry, and intersection lane configuration. Turn movement counts were extracted from the videotapes recorded during the field study. These counts were produced for the upstream and downstream intersections for all traffic movements during each 15-minute interval in the study period. Travel time, percent stopping, and control delay data were collected for one direction of travel on each segment. The geometric data were collected during a site survey conducted prior to the start of the field study.
Signal timing data were obtained from the local agency for the period corresponding to the time of the field study. These data included the signal controller settings, detection design, and time-of-day signal timing plans. The signal controller settings consisted of the cycle length, minimum and maximum green interval durations, detection input settings, and coordination settings.

The data collected during the field studies are summarized in Table B-6. The statistics listed represent average values for the 16 fifteen-minute intervals that occurred during the four-hour study period. The average speed listed in column 4 is computed as a space mean speed (i.e., the average travel time divided into the segment length). In general, the average speed is between 70 and 80 percent of the speed limit. However, the average speed at Pratt Street is much smaller than the speed limit and reflects the dominance of the signal control delay in the segment travel time, relative to the time required to traverse the short segment length.

**TABLE B-6  Data reduction statistics for full segment study sites**

<table>
<thead>
<tr>
<th>Site</th>
<th>Street Segment</th>
<th>Average Volume¹, veh/h</th>
<th>Travel Speed², mph</th>
<th>Signal Timing³</th>
<th>Performance³</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Cycle Length, s</td>
<td>Green/Cycle Ratio</td>
</tr>
<tr>
<td>1</td>
<td>Aviation Parkway</td>
<td>920</td>
<td>47</td>
<td>80</td>
<td>0.63</td>
</tr>
<tr>
<td>2</td>
<td>S.E. McLoughlin Boulevard</td>
<td>1487</td>
<td>33</td>
<td>85</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>75</td>
<td>0.56</td>
</tr>
<tr>
<td>3</td>
<td>S.W. Barbur Boulevard</td>
<td>476</td>
<td>29</td>
<td>100</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>120</td>
<td>0.50</td>
</tr>
<tr>
<td>4</td>
<td>S.E. Powell Boulevard</td>
<td>1644</td>
<td>23</td>
<td>85</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>120</td>
<td>0.64</td>
</tr>
<tr>
<td>5</td>
<td>E. Speedway Boulevard</td>
<td>1447</td>
<td>24</td>
<td>90</td>
<td>0.66</td>
</tr>
<tr>
<td>6</td>
<td>Pratt Street</td>
<td>1410</td>
<td>8</td>
<td>110</td>
<td>0.48</td>
</tr>
</tbody>
</table>

Notes:
1 - Average volume exiting the segment at the downstream intersection (sum of all exiting movements).
2 - Travel speed of through-through vehicles. These vehicles entered the segment and exited the segment as a through vehicle. This speed is computed as a space-mean speed.
3 - Green/cycle length ratio, percent stopping, and control delay for the through lane group at the downstream intersection approach in the subject direction of travel. Where two rows of data are provided for a given site, the top row corresponds to the off-peak traffic period and the bottom row corresponds to the peak traffic period.

**Model Calibration**

This section describes the findings from the model calibration process. The calibration process was based on the specification of a model form with selected calibration coefficients. Regression techniques were then used to find the coefficients that provided the best-fit to the data.
The cycle-based running speeds collected at each of the ten partial segment study sites represent the dependent variable for model calibration. The analysis initially focused on a model that included the effect of flow rate, segment length, percent heavy vehicles, and the percentage of through vehicles. A through vehicle was defined as a vehicle that entered the segment at the upstream signalized intersection and exited the segment at the downstream signalized intersection.

A series of regression analyses revealed that flow rate was the only variable that had a statistically significant effect on running speed. A correlation with segment length was evident in a graphical examination of the data, but the corresponding regression coefficient was not statistically significant. It was rationalized that the 10-site database was too small to adequately capture the subtle effect of segment length on speed. It was further rationalized that the effect of length is likely to exist given that it is documented in Chapter 10 of the HCM (5) and has been confirmed by research conducted by Prassas (1, 2). For these reasons, the segment length effect represented in the Prassas model (as described in a previous part of this appendix) was retained in the regression model.

The percentage of heavy vehicles ranged from 0 to 8 percent, with an average of 4.0 percent. An initial model form that included an adjustment factor for heavy vehicle percentage indicated that heavy vehicles have a running speed that is 3.0 percent slower than that for passenger cars. However, this reduction in speed was not practically significant for the percentage of heavy vehicles in the typical urban arterial traffic stream (i.e., it translated into about a 0.1 percent speed reduction for a traffic stream with 4.0 percent heavy vehicles). Moreover, its effect was not found to be statistically significant so this variable was removed from the model.

The percentage of through-through vehicles averaged 76 percent, with 70 percent of the observations ranging from 60 to 95 percent through-through vehicles. An initial model form that included an adjustment factor reflecting this percentage revealed that exclusive through-through traffic streams had about 1.0 percent higher speed than non-through-through streams. As with the heavy vehicle influence, this effect was found to be neither practically nor statistically significant and was removed from the model.

The best-fit speed-flow model was specified using the following equation:

$$S_v = S_{fe} f_L f_n$$

with,

$$S_{fe} = b_0 + \sum_{i=1}^{9} b_i Site_i$$

$$f_L = 1.02 - 4.7 \frac{S_{fe} - 19.5}{5280} L$$

$$f_n = \frac{1}{2} \left(1 + \left[1 - \frac{v}{52.8 S_{fe} f_L} \right]^{b_{10}}\right)$$

with, $b_0$, $b_i$, and $b_{10}$ being constants.
where,

\( S_v \) = running speed based on volume, mph;

\( S_{fo} \) = base free-flow speed, mph;

\( f_L \) = segment length adjustment factor;

\( f_{iv} \) = inverse proximity adjustment factor;

\( b_i \) = calibration coefficient \((i = 0, 1, 2, \ldots, 10)\);

\( Site_i \) = indicator variable for site \(i\) (= 1.0 if data corresponds to site \(i\), 0.0 otherwise);

\( L \) = segment length, mi; and

\( v \) = flow rate, veh/h/ln.

The nonlinear regression procedure (NLIN) provided in the SAS system was used to estimate the best-fit regression coefficients. This procedure uses an iterative search method to adjust model coefficients and, thereby, optimize the fit of non-linear regression models. The statistics related to the calibrated model are shown in Table B-7. The calibrated coefficient values shown in the bottom half of the table can be used in Equations 13 to 16 to estimate the running speed of an urban street traffic stream. The \( R^2 \) for the model is 0.89, indicating that the model explains 89 percent of the variability in the data. The standard deviation of the model estimate is 1.5 mph.

### TABLE B-7 Speed-flow model statistics

<table>
<thead>
<tr>
<th>Model Statistics</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( R^2 ):</td>
<td>0.89</td>
</tr>
<tr>
<td>Observations ( n_i ):</td>
<td>200 signal cycles (6758 vehicles)</td>
</tr>
<tr>
<td>Standard Deviation ( s_v ):</td>
<td>±1.5 mph</td>
</tr>
</tbody>
</table>

#### Range of Model Variables

<table>
<thead>
<tr>
<th>Variable</th>
<th>Variable Name</th>
<th>Units</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>( v )</td>
<td>Flow rate</td>
<td>veh/h/ln</td>
<td>61</td>
<td>610</td>
</tr>
<tr>
<td>( L )</td>
<td>Segment length</td>
<td>miles</td>
<td>0.19</td>
<td>0.70</td>
</tr>
</tbody>
</table>

#### Calibrated Coefficient Values

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
<th>Value</th>
<th>Std. Dev.</th>
<th>t-statistic</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b_0 )</td>
<td>Base free-flow speed for Site 10</td>
<td>40.92</td>
<td>0.229</td>
<td>178.6</td>
</tr>
<tr>
<td>( b_1 )</td>
<td>Added speed for Site 1</td>
<td>0.12</td>
<td>0.275</td>
<td>0.4</td>
</tr>
<tr>
<td>( b_2 )</td>
<td>Added speed for Site 2</td>
<td>0.38</td>
<td>0.278</td>
<td>1.4</td>
</tr>
<tr>
<td>( b_3 )</td>
<td>Added speed for Site 3</td>
<td>7.40</td>
<td>0.303</td>
<td>24.5</td>
</tr>
<tr>
<td>( b_4 )</td>
<td>Added speed for Site 4</td>
<td>-0.58</td>
<td>0.262</td>
<td>-2.2</td>
</tr>
<tr>
<td>( b_5 )</td>
<td>Added speed for Site 5</td>
<td>3.07</td>
<td>0.238</td>
<td>12.9</td>
</tr>
<tr>
<td>( b_6 )</td>
<td>Added speed for Site 6</td>
<td>9.98</td>
<td>0.262</td>
<td>38.1</td>
</tr>
<tr>
<td>( b_7 )</td>
<td>Added speed for Site 7</td>
<td>-4.86</td>
<td>0.379</td>
<td>-12.8</td>
</tr>
<tr>
<td>( b_8 )</td>
<td>Added speed for Site 8</td>
<td>-4.38</td>
<td>0.334</td>
<td>-13.1</td>
</tr>
<tr>
<td>( b_9 )</td>
<td>Added speed for Site 9</td>
<td>6.28</td>
<td>0.271</td>
<td>23.2</td>
</tr>
<tr>
<td>( b_{10} )</td>
<td>Calibration coefficient for flow rate effect</td>
<td>0.21</td>
<td>0.028</td>
<td>7.6</td>
</tr>
</tbody>
</table>

The regression coefficients for the calibrated model are listed in the last rows of Table B-7. The coefficients \( b_0 \) through \( b_9 \) indicate the base free-flow speed for each of the 10 sites. The
intercept coefficient $b_0$ represents the base free-flow speed for Site 10. The speed for the other nine sites is computed by adding the site’s coefficient with that for Site 10. Thus, the base free-flow speed for Site 6 is 50.90 mph (= 40.92 + 9.98).

The last coefficient listed in Table B-7 represents the effect of flow rate. As noted in a previous part of this appendix, the value of this coefficient should be 0.5 on a theoretic basis. Deviation from this value was allowed as a calibration adjustment to improve the fit of the model to the data. Values less than 0.5 flatten the slope of the speed-flow relationship. That is, the change in speed for a specified change in volume decreases as the value is reduced from 0.5.

One means of assessing the model’s fit is through a graphical comparison of the observed and predicted running speeds. This comparison is provided in Figure B-8. The trend line in this figure does not represent the line of best fit; rather, it is a “$y = x$” line. The data would lie on this line if the model predictions exactly equaled the observed data. The clustering of the data around this line indicates that the model is able to predict running speed without bias.

The fit of the calibrated model to the running speeds observed at three sites is shown in Figure B-9. Similar trends were found for the other sites; only three sites are shown in the figure to improve the clarity of the presentation. Each data point represents the observed flow rate and running speed for one signal cycle. All three sites shown indicate a slight reduction in speed with an increase in flow rate. The backward projection of each trend line to its intercept with the zero flow rate axis yields the free-flow speed for the site.
The database used to calibrate the base free-flow speed model included the data from the ten partial segment study sites described in the previous section. It also included the data collected by Fitzpatrick et al. (3). The combined database included data for 50 sites, five of which are classified as collector streets and the balance of which are classified as arterial streets.

The data collected by Fitzpatrick et al. (3) represent a mix of arterial and collector streets. The database includes sites with curb as well as sites with shoulder treatments. The number of through lanes ranges from two to six (1 to 3 in each direction). The number of segments with no median, non-restrictive median, and restrictive median is equally distributed. All segments have a length of 0.4 mi or more, such that the effect of segment length is negligible. A vehicle was defined by Fitzpatrick et al. to be freely-flow if its headway exceeded 5.0 s and its tailway exceeded 3.0 s. For each site, the average spot speed was converted into a space-mean speed estimate using the following equation (12):

\[
S_{sms} = S_{spd} - \frac{\sigma_{spd}^2}{S_{spd}}
\]  \hspace{1cm} (17)

where,

- \(S_{sms}\) = estimated space-mean speed, mph.

The base free-flow speeds for the 10 partial segment study sites were obtained from the speed-flow model regression coefficients in Table B-7. These coefficients are based on space-mean speed estimates and, by the nature of their derivation, represent the base free-flow speed at each of the ten study sites.

Figure B-9. Effect of flow rate on running speed.

Base Free-Flow Speed Model
The analysis initially focused on a model that included the effect of speed limit, grade, access point density, curb presence, and median type. Access point density was based on the count of all access points, regardless of their usage (i.e., residential, commercial, or business). The analysis indicated that the effect of access point density diminished with an increase in the number of through lanes. Further exploration indicated that the ratio of the access point density to the number of through lanes provided an improved fit over the use of access point density alone.

A series of regression analyses revealed that speed limit, access point density, curb presence, and median type were correlated with free-flow speed. The effect of grade suggested that speed decreased about 0.4 mph for each 1.0 percent grade. Thus, a segment with an uphill grade of 2.0 percent would have a base free-flow speed that is 0.8 mph slower than a similar street on level terrain. However, the standard deviation associated with this coefficient was 0.6, which is quite large relative to the coefficient value and, thus, the coefficient is not statistically significant. For this reason, the grade variable was removed from the model.

The effect of median type considered various combinations of the three median types found at the sites (i.e., no median, non-restrictive, and restrictive). The percentage of the segment length with a restrictive median $P_{rm}$ was found to be the most informative and statistically significant variable. The percentage $P_{rm}$ for segments with a restrictive median tended to be 74 percent or more, depending on the number of median openings. The value of $P_{rm}$ equaled 0.0 for those sites with either non-restrictive or no median for their full length.

The analysis indicated that the presence of a curb tended to reduce the base free-flow speed. However, this effect varied, depending on whether the curb was adjacent to the outside lane or on both the outside and inside lanes. To quantify this influence, a variable for curb presence was included in the model along with a variable for curb-median interaction. The influence of a curb adjacent to the inside lane only was not tested because none of the sites in the database had this type of cross section.

The best-fit base free-flow speed model was specified using the following equation:

$$S_{fo} = c_0 + c_1 S_{pl} + c_2 P_{rm} + c_3 \frac{D_a}{N} + c_4 I_{curb} + c_5 I_{curb} P_{rm}$$

where,
- $S_{fo}$ = base free-flow speed, mph;
- $c_i$ = calibration coefficient ($i = 0, 1, 2$);
- $S_{pl}$ = posted speed limit, mph;
- $P_{rm}$ = percent of segment length with restrictive median, percent;
- $D_a$ = access point density (total access points on both sides of street), points/mi;
- $N$ = number of through traffic lanes in subject direction of travel; and
- $I_{curb}$ = indicator variable for presence of curb (= 1.0 if curb is present on the right-hand side of the traveled way, 0.0 otherwise).

The linear regression procedure provided in the SAS system was used to estimate the best-fit regression coefficients. This procedure uses the least-squares method to optimize the fit of the regression model. The weighted regression option was used to properly reflect the varying
uncertainty associated with each estimate of base free-flow speed. This uncertainty was quantified using the standard deviation associated with each regression coefficient for the partial segment sites and the standard deviation of the mean for each of the sites from the Fitzpatrick et al. (3) database.

The statistics related to the calibrated model are shown in Table B-8. The calibrated coefficient values shown in the bottom half of the table can be used in the model to estimate the base free-flow speed of an urban street. The $R^2$ for the model is 0.81, indicating that the model explains 81 percent of the variability in the data. The standard deviation of the model estimate is 2.4 mph.

The regression coefficients for the calibrated model are listed in the last rows of Table B-5. The t-statistic for each coefficient is also provided with each variable. In general, a t-statistic whose absolute value exceeds 2.0 is considered significant at a 5 percent level of confidence. The variables associated with “percent restrictive median” and “curb presence” do not reach this level of confidence. However, their coefficient values are reasonable and their interaction term is significant. For these reasons, they were kept in the model.

The curb and median-presence variables, as well as their interaction variable, provide an interesting result. Individually, they indicate that the presence of a curb on the right side of the traveled way results in speed reduction of 0.47 mph and the presence of a restrictive median results in a speed increase of 1.5 mph. However, when curb and a restrictive median are present (i.e., a raised-curb median), the net effect is a speed reduction on the order of about 2.7 mph. Thus, when there is no curb and a depressed median is present, the net effect is a 1.5 mph speed increase.

<table>
<thead>
<tr>
<th>TABLE B-8  Base free-flow speed model statistics</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Model Statistics</strong></td>
</tr>
<tr>
<td>$R^2$:</td>
</tr>
<tr>
<td>Observations $n$:</td>
</tr>
<tr>
<td>Standard Deviation $s_e$:</td>
</tr>
</tbody>
</table>

**Range of Model Variables**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Variable Name</th>
<th>Units</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{pl}$</td>
<td>Speed Limit</td>
<td>veh/h</td>
<td>30</td>
<td>55</td>
</tr>
<tr>
<td>$P_{rm}$</td>
<td>Percent of length with restrictive median</td>
<td>%</td>
<td>0</td>
<td>92</td>
</tr>
<tr>
<td>$N$</td>
<td>Number of through lanes in one direction</td>
<td>--</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>$D_{a}$</td>
<td>Access point density</td>
<td>points/mi</td>
<td>0</td>
<td>103</td>
</tr>
</tbody>
</table>

**Calibrated Coefficient Values**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
<th>Value</th>
<th>Std. Dev.</th>
<th>t-statistic</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_0$</td>
<td>Intercept</td>
<td>25.6</td>
<td>3.3</td>
<td>7.7</td>
</tr>
<tr>
<td>$c_1$</td>
<td>Effect of speed limit</td>
<td>0.47</td>
<td>0.072</td>
<td>6.5</td>
</tr>
<tr>
<td>$c_2$</td>
<td>Effect of percent restrictive median</td>
<td>0.015</td>
<td>0.014</td>
<td>1.1</td>
</tr>
<tr>
<td>$c_3$</td>
<td>Effect of access point density</td>
<td>-0.078</td>
<td>0.020</td>
<td>-3.9</td>
</tr>
<tr>
<td>$c_4$</td>
<td>Effect of curb presence</td>
<td>-0.47</td>
<td>0.960</td>
<td>-0.5</td>
</tr>
<tr>
<td>$c_5$</td>
<td>Interaction of curb and median presence</td>
<td>-0.037</td>
<td>0.017</td>
<td>-2.2</td>
</tr>
</tbody>
</table>
One means of assessing the model’s fit is through a graphical comparison of the observed and predicted base free-flow speeds. This comparison is provided in Figure B-10. The trend line in this figure does not represent the line of best fit; rather, it is a “y = x” line. The data would lie on this line if the model predictions exactly equaled the observed data. The clustering of the data around this line indicates that the model is able to predict base free-flow speed without bias.

Figure B-10. Comparison of measured and predicted base free-flow speed.

Residual Start-Up Lost Time Model

The database used to calibrate the residual start-up lost time model included the data from the six full-segment study sites described in the previous section. The calibration data include the average running time $T_R$ for each segment and the running time predicted using the calibrated free-flow speed model and the speed-flow model. The residual start-up lost time was computed using the following equation:

$$T_x = T_R - \frac{3600 L}{5280 S_f} f_v$$  \hspace{1cm} (19)

The average running time for each site was computed using the travel time data for those vehicles that did not stop at the downstream signalized intersection.

Figure B-11 illustrates the relationship between computed residual start-up lost time and segment length for the six study sites. Each data point shown in the figure corresponds to one site. The trend in the data is similar to that observed in Figure B-4. Specifically, the residual start-up lost time decreases from about 4.0 s to -1.0 s as segment length increases from 0.1 to 0.6 mi. The thick trend line shown in the figure represents the equation shown previously in Figure B-4.
Figure B-11. Residual start-up lost time observed at six study sites.

It was rationalized that the equation calibrated using the data in Figure B-4 is likely to be more accurate than an equation developed from the six study sites, especially given the highly variable nature of the data found in these sites. A comparison of this equation with the data in Figure B-11 indicates that the equation reasonably tracks the data. Hence, the equation shown in Figure B-11 is recommended for use in computing residual start-up lost time.

REFERENCES


APPENDIX C

Procedure for Estimating Arrival Flow Profile
APPENDIX C

PROCEDURE FOR ESTIMATING ARRIVAL FLOW PROFILE

INTRODUCTION

This appendix describes a procedure for predicting the arrival flow profile for an intersection approach. This profile describes the variation in flow rate during the average signal cycle, as it would be measured a specified point downstream of a signalized intersection. This profile can be used to estimate the arrival flow profile on the approach to a downstream unsignalized or signalized intersection. For an unsignalized intersection, it can be used to estimate the proportion of time that a traffic platoon is present in the intersection and, thereby, blocking the minor traffic movements at this intersection. For a signalized intersection, it can be used to estimate the proportion of vehicles that arrive at the downstream intersection during a specified time period. The proportion can be used, with information about the downstream intersection’s signalization, to estimate the delay to the arriving vehicles. The procedure description is lengthy because of its many elements; however, it is not computationally complex and is easily implemented in software. Its input variables are the same as those of the urban streets methodology in Chapter 15 of the *Highway Capacity Manual (HCM)* (1). However, it also requires signal offset, upstream signal timing, and an origin-destination volume distribution.

This appendix consists of four main parts. The first part describes a platoon dispersion model. The second part describes a procedure for determining the arrival flow profile at a specified point downstream of a signalized intersection. The procedure includes equations for estimating the proportion of time that a platoon is present (i.e., the “proportion of time blocked”) and the proportion of arrivals during the effective green time for the downstream through lane group (i.e., the “proportion of vehicles arriving on green”). The third part describes the findings from a sensitivity analysis using the procedure. The last part summarizes the findings from the calibration of selected model components.

BACKGROUND

This part of the appendix consists of three sections. The first section describes a platoon dispersion model. The calibration of this model to various urban street conditions has been the subject of investigation by several researchers. The second section summarizes the findings of these researchers. The last section describes a procedure for using the platoon dispersion model to estimate the arrival flow profile and to compute key descriptive statistics (i.e., “proportion of time blocked” and “proportion of vehicles arriving on green”).

Platoon Dispersion Model

This section describes a model for predicting the extent of platoon dispersion as a function of segment running time. The model was originally developed by Robertson (2). It is based on the division of the signal cycle into an integer number of intervals, each with an equal duration, called
time steps. Input to the model is the flow profile for a specified traffic movement discharging from an upstream signalized intersection, as defined in terms of the flow rate for each time step. Output from the model include: (1) the arrival time of the leading vehicles in the platoon to a specified downstream intersection and (2) the flow rate for each time step at this intersection. In general, the flow profile at the downstream intersection has a lower peak flow rate and its flow is spread out over a longer period of time, relative to the input flow profile. These characteristics describe the dispersion of platoon vehicles. The rate of dispersion increases with increasing variance in the speed distribution, as may reflect the turbulence caused by access point activity or by on-street parking maneuvers.

The platoon dispersion model is described by the following equation:

\[ q'_{abj} = F q'_{ki} + (1 - F) q'_{abj-1} \]  
(1)

with,

\[ j = i + t' \]  
(2)

where,

- \( q'_{abj} \) = arrival flow in time step \( j \) at a downstream intersection from upstream source \( u \), veh/step;
- \( q'_{ui} \) = discharge flow in time step \( i \) at upstream source \( u \), veh/step;
- \( F \) = smoothing factor;
- \( j \) = time step associated with platoon arrival time; and
- \( t' \) = platoon arrival time, steps.

Figure C-1 illustrates the arrival flow profile obtained from Equation 1. In this figure, the departure flow profile is input to the model as variable \( q'_{ui} \). The dashed rectangles that form the departure flow profile indicate the flow rate during each of nine time steps (\( i = 1, 2, 3, \ldots 9 \)) that are each \( dt \) seconds in duration. The vehicles that depart in the first time step (\( i = 1 \)) arrive at the downstream intersection after traveling an amount of time equal to \( t' \) steps. The arrival flow at any time step \( j \) (= \( i + t' \)) is computed using Equation 1.

Alternative models of platoon dispersion have been proposed (3, 4, 5). Seddon (6) and Tracz (5) conducted comparative evaluations of these models based on consideration of accuracy, complexity, and computational efficiency. Their findings indicate that Equation 1 consistently performed well. None of the evaluated models provided better performance than Equation 1.

**Dispersion Model Calibration Coefficients**

This section examines the equations used to predict the smoothing factor \( F \) and the platoon arrival time \( t' \) in Equation 1. These equations include theoretic constructs and calibration coefficients that are intended to provide accurate flow profile predictions when Equation 1 is applied to an arterial street segment.
Seddon (7) demonstrated that underlying Equation 1 is a geometric distribution that describes the probability of a vehicle’s arrival downstream during any given time step. He derived the following relationship between the smoothing factor and the average increase in platoon running time due to dispersion:

$$ F = \frac{1}{1 + u'} \quad (3) $$

where,

$$ u' = \text{average increase in platoon running time, steps.} $$

By definition, the sum of $u'$ and the platoon arrival time $t'$ equals the average running time $T'_r$ (i.e., $T'_r = u' + t'$). This relationship is shown in Figure C-2. When substituted in Equation 3, it yields the following equation.

$$ F = \frac{1}{1 + T'_r - t'} \quad (4) $$

where,

$$ T'_r = \text{average running time, steps.} $$

Based on empirical evidence, Robertson (2) offered that platoon arrival time $t'$ could be estimated as a proportion of the average running time $T'_r$, using the following equation.

$$ t' = \beta T'_r \quad (5) $$

where,

$$ \beta = \text{platoon arrival time calibration coefficient.} $$
Robertson (2) recommended the use of $\beta = 0.80$ in Equation 5 to estimate the platoon arrival time. In a review of the literature related to this coefficient, Axhausen and Korling (8) reported that many researchers found $\beta$ to vary on a site-by-site basis, with values ranging from 0.59 to 0.99.

Combining the relationship for $\beta$ with Equation 4 yields the following equation.

$$F = \frac{1}{1 + (1 - \beta) T'_r}$$  \hspace{1cm} (6)

An alternative form of Equation 6 was developed by Robertson (2) and is commonly found in the literature, it is:

$$F = \frac{1}{1 + \alpha \beta T'_r}$$  \hspace{1cm} (7)

where,  

$\alpha =$ platoon dispersion calibration coefficient.

Equation 7 has two calibration coefficients in its denominator, as opposed to the one coefficient in Equation 6.
Based on empirical evidence, Robertson (2) offered that a dispersion coefficient \( \alpha \) of 0.50 provided the best agreement with data collected at four sites, when \( \beta \) was set to 0.80. The review by Axhausen and Korling (8) revealed that \( \alpha \) also varied on a site-by-site basis, with values ranging from 0.10 to 0.63. It should be noted that these values of \( \alpha \) were based on \( \beta \) being set to 0.80.

In separate research projects, Lam (9) and McCoy et al. (10) used field-measured flow profiles from nine sites to calibrate Equations 5 and 7. Specifically, they used Equation 1 with Equations 5 and 7 to determine the values of both \( \alpha \) and \( \beta \) that provided the most accurate estimate of the downstream flow profile. The values of the calibration coefficients \( \alpha \) and \( \beta \) varied widely among sites but their product (\( \alpha \beta \)) was fairly stable and ranged from 0.13 to 0.25. This finding suggests that Equation 6, with a single calibration coefficient, may be a more appropriate form than Equation 7.

**Arrival Flow Profile Prediction Procedure**

This section describes a procedure for using a platoon dispersion model to estimate the arrival flow profile at an intersection downstream of a signalized intersection. The procedure includes equations to estimate key descriptive statistics (i.e., “proportion of time blocked” and “proportion of vehicles arriving on green”). These statistics can be used with the procedures in Chapters 16 and 17 of the *HCM* to estimate the delay incurred by selected traffic movements at a downstream intersection.

Figure C-3 illustrates how the platoon dispersion model can be used to estimate the flow profile at any specified intersection that is downstream of a signalized intersection. Typically, there are three upstream traffic movements that depart at different times during the signal cycle, they are: cross-street right turn, major-street through, and cross-street left turn. Traffic may also enter the segment at various mid-segment access points. These movements combine to form an arrival flow profile downstream.

In application, the departure flow profile is first derived for each movement. These profiles are shown in the first of the three x-y plots in Figure C-3. Then, a platoon dispersion model is used to estimate the arrival flows for each movement at a downstream intersection. These arrival flow profiles are shown in the second x-y plot in Figure C-3. Finally, these three arrival flow profiles are added together to produce the combined arrival flow profile. This profile is shown in the third x-y plot in Figure C-3. The validity of this combination technique was confirmed by Tarnoff and Parsonson (11); it is used in the TRANSYT-7F software program (12) to estimate arrival flow profiles for signal system evaluation.

Tarnoff and Parsonson (11) also investigated the effect of mid-segment access point traffic on the accuracy of the aforementioned arrival flow profile estimation procedure. Specifically, their investigation examined whether the periodic arrival of platoons at unsignalized access points tended to meter access point vehicle entry. They pointed out that the TRANSYT-7F software does not include this sensitivity. Rather, TRANSYT-7F uniformly distributes the net inflow (i.e., flow for all mid-segment access points combined) among all time steps. Tarnoff and Parsonson (11) found that, for typical access point volumes, a more refined approach for modeling the combined access
point volume did not improve profile estimation accuracy. This finding suggests that the TRANSYT-7F approach yields satisfactory accuracy.

![Diagram showing arrival flow profile estimation procedure.](image)

**Figure C-3. Arrival flow profile estimation procedure.**

The arrival flow profile can be used for both unsignalized and signalized intersection capacity analysis. For unsignalized intersection analysis, the flow profile is used to estimate the time that a dense platoon is present in the intersection (i.e., total time blocked $t_p$ when the units are seconds, or $t'_p$ when the units are time steps). During this time period, the platoon is sufficiently dense as to preclude a minor movement driver from finding an acceptable gap.

The use of the arrival flow profile to estimate the blocked period duration is shown in Figure C-4. The profile shown represents the sum of the arrival flows for each upstream intersection movement. The dashed line represents the critical platoon flow rate $q_c$. Flow rates in excess of this
threshold are rationalized to be associated with platoon headways that are too short to be entered (or crossed) by minor movements at an unsignalized intersection. A critical platoon flow rate of 1000 veh/h was recommended by Bonneson and Fitts (13) for left turns from the major street. Other rates can be rationalized for the left-turn, through, and right-turn movements from the minor street.

![Combined Arrival Flow Profile for Through Lane Group](image)

**Figure C-4.** Use of arrival flow profile to estimate the blocked period duration.

The flow shown in Figure C-4 represents the sum of the arrival flow profiles for each of the upstream intersection movements after they are factored downward to reflect their contribution to the subject downstream lane group that conflicts with the minor movement (typically, the through lane group). The nature of this adjustment is noted in the equation in the figure, where the flow in each upstream movement’s arrival profile is multiplied by the proportion of vehicles that depart the upstream intersection for the subject lane group.

In the situation where a driver desires to complete a left turn from the major street across the traffic stream represented by Figure C-4, the proportion of time blocked is computed using the following equation.

$$ p = \frac{t_p' \cdot d_i}{C} $$

where,

$p$ = proportion of time blocked;
$t_p'$ = blocked period duration, steps;
$d_i$ = time step duration, s/step; and
$C$ = cycle length, s.

Other variations of this equation, as they relate to the minor street movements, are described in Chapter 17 of the HCM.
For signalized intersection capacity analysis, the flow profile can be used to estimate the proportion of vehicles that arrive during the effective green period. The effective green period is typically that associated with the through phase; however, it could also be that associated with the left-turn phase. The manner in which this proportion is estimated from the arrival flow profile is shown in Figure C-5. Like that shown in Figure C-4, the flow in Figure C-5 represents the sum of the arrival flow profiles for each of the upstream intersection movements after they are factored downward to reflect their contribution to the subject downstream lane group (typically, the through lane group).

![Combined Arrival Flow Profile for Through Lane Group](image)

\[ V_g = \text{arrivals during effective green, veh} \]

\[ V_d = \text{lane group volume, veh/h} \]

\[ P = \frac{V_g}{V_d \cdot C} \]

\[ (9) \]

**DETERMINING ARRIVAL FLOW PROFILE**

This part of the appendix describes a procedure for determining the arrival flow profile at a specified point downstream of a signalized intersection. The procedure includes equations for estimating key statistics, such as the proportion of time that a platoon is present (i.e., the “proportion
of time blocked”) and the proportion of arrivals during the effective green time for the downstream through lane group (i.e., the “proportion of vehicles arriving on green”). The “proportion of time blocked” can be used with the procedure in Chapter 17 of the HCM to estimate the capacity and delay to one or more minor movements at an unsignalized intersection, as may be influenced by an upstream signalized intersection. The “proportion of vehicles arriving on green” can be used with the procedure in Chapter 16 of the HCM to estimate the progression adjustment factor $PF$ and the platoon ratio $R_P$.

The procedure for determining the arrival flow profile and associated statistics is described in three sections. The first section describes the sequence of calculations needed to estimate the arrival flow profile for each upstream traffic movement. These calculations must always be completed, regardless of which key statistic is sought. The second section describes the sequence of calculations needed to estimate the proportion of time blocked. The last section describes the sequence of calculations needed to estimate the proportion of vehicles arriving on green and associated statistics.

**Movement Arrival Flow Profiles**

This section describes the sequence of calculations needed to estimate the arrival flow profile. It consists of four sections. The first section identifies the data required to implement the procedure. The second section describes the equations used to compute the volume for the upstream traffic movements. The third section describes the computations needed to determine the discharge flow profile. The last section describes the use of the platoon dispersion model to estimate the arrival flow profile for each upstream traffic movement.

**Data Requirements**

**Travel Distance ($D$).** This variable defines the distance from the upstream intersection to a specified downstream location. This location could be an unsignalized intersection or the first downstream signalized intersection. If the desired location is a signalized intersection, then the travel distance equals the segment length $L$.

**Running Speed ($SR$).** This variable represents the segment length $L$ divided by the segment running time $TR$, as determined using the procedure described in Appendix B.

**Maximum Platoon Headway ($t_b$).** Threshold headway used to define when a platoon is sufficiently dense as to prevent crossing or entry maneuvers by minor traffic movements at an unsignalized intersection. A value of 3.6 s/veh is recommended unless field observation suggests that another value is more appropriate.

**Time Step Duration ($dt$).** Flow profile estimation requires division of the signal cycle into an integer number of time steps, each of equal duration. Typical time step durations range from 1 to 3 s/step with larger values in this range used for longer cycle lengths.
**Relative Offset \((O)\).** The time duration measured from the start of the major-street through movement green interval at the upstream intersection to the start of the major-street through movement green interval at the downstream intersection. For semi-actuated signal systems, the offset is likely to be referenced to the end of the through green (or yellow change) interval.

Offset information is needed only when the proportion of vehicles arriving on green is desired. For planning applications, it is reasonable to assume that the relative offset in the peak travel direction is equal to the average running time.

**Start of Green \((t_{G,u})\).** The time corresponding to the start of the green interval for the phase that serves upstream intersection traffic movement \(u\) (where, \(u\) = subscript designating upstream entry traffic movement, 1 = left-turn from cross street, 2 = through from major street, 3 = right-turn from cross street, 4 = mid-segment entry between the upstream signalized intersection and the specified downstream intersection). A time is not provided for the mid-segment entry.

Based on the definition of relative offset, the start of green for the phase serving the upstream through movement is 0.0 s. The start of green value for each movement must be consistent with the phase sequence. If an upstream cross-street turn movement is served with its adjacent through movement, then the start of green value should be that of the adjacent through movement.

Start-of-green information is needed only when the proportion of vehicles arriving on green is desired. For planning applications, it is reasonable to use default \(g/C\) ratios to define the phase duration and start-of-green for the upstream traffic movements.

**Phase Duration \((G_u + Y_u)\).** The combined duration of the green, yellow change, and red clearance intervals for the phase that serves upstream movement \(u\). Variable \(G_u\) represents the green interval duration and \(Y_u\) represents the sum of the yellow change and red clearance intervals. The phase duration must be consistent with the start-of-green information.

If an upstream cross-street turn movement is served with its adjacent through movement, then the phase duration should be equal to that of the adjacent through movement. For semi-actuated signal systems, the green interval duration is likely to vary with each signal cycle. In this situation, the “average” green duration for each signal phase must be estimated using the procedure described in Appendix D.

Phase-duration information is needed only when the proportion of vehicles arriving on green is desired. For planning applications, it is reasonable to use default \(g/C\) ratios to define the phase duration and start-of-green for the upstream traffic movements.

**Discharge Flow Rate \((q_{iu})\).** For upstream traffic movement \(u\) served by an exclusive lane (or lanes), the discharge flow rate is equal to the saturation flow rate obtained from the Volume Adjustment and Saturation Flow Rate Worksheet (and expressed in units of \(\text{veh/h/ln}\)). If an upstream cross-street turn movement shares a lane with its adjacent through movement, then the discharge flow rate for the turn movement can be estimated using the following equation:
with,
\[ s_{qu} = \frac{1800 \, P_T}{1 + P_T(E_T - 1)} \]  \hspace{1cm} (10)

\[ P_T = P_t \, n_t \leq 1.0 \]  \hspace{1cm} (11)

where,
- \( s_{qu} \): shared lane discharge flow rate for upstream traffic movement \( u \), veh/h/ln;
- \( P_T \): proportion of turning vehicles in the shared lane;
- \( P_t \): proportion of turning vehicles in the through lane group that shares a lane with the subject turn movement;
- \( n_s \): number of lanes in the through lane group that shares a lane with the subject turn movement; and
- \( E_T \): through-car equivalent for the turn movement.

When Equation 10 is applied to a left-turn movement, the through-car equivalent variable can be obtained from Exhibit C16-3 in Chapter 16 of the HCM. This exhibit indicates that \( E_T \) equals 1.4, 2.5, and 4.5, for opposing flows of 1, 600, and 1200 veh/h, respectively. When Equation 10 is applied to a right-turn movement, \( E_T \) equals 1.18.

Discharge information is needed only when the proportion of vehicles arriving on green is desired. For planning applications, it is reasonable to use default saturation flow rates and turn percentages for typical upstream traffic movements. The accuracy of the flow profile is not highly sensitive to the value of the discharge flow rate. Hence, a discharge flow rate that is accurate to the nearest 100 veh/h/ln should be adequate for flow profile estimation.

**Lanes Serving Movement (n_u).** This variable represents the number of lanes serving upstream traffic movement \( u \). If the upstream movement is a turn from a shared lane, then the number-of-lanes \( n \) is equal to 1.0. Turn movements entering at a mid-segment entry point also have a number-of-lanes equal to 1.0.

**Exit Lane Group Volume (V_{d}).** This variable represents the volume for lane group \( d \) that exits the segment at the specified downstream intersection. Also includes the aggregate volume of traffic exiting the segment via one or more mid-segment access points. Subscript \( d \) equals 1, 2, 3, or 4; where 1 = left-turn lane group, 2 = through lane group, 3 = right-turn lane group, and 4 = mid-segment exit between the upstream signalized intersection and the specified downstream intersection. Note, the “.” in the variable name means the variable is summed for all corresponding subscript values.

**Proportion of Downstream Volume from Upstream Movement (P_{u,d}).** This proportion represents the distribution of a lane group’s volume to each of the upstream traffic movements. Four proportions must be specified for each lane group. Three proportions apply to the upstream phases serving the left-turn, through, and right-turn movements. The fourth proportion represents the flow contribution from all mid-segment access points combined. The four proportions must sum to 1.0 for each lane group.
If a turn movement at the upstream intersection is served by a phase but this movement can also pass through the intersection permissively (e.g., right-turn on red, protected/permissive left-turn phasing) then the proportion \( P_{u,d} \) should reflect only the volume served by the parent upstream phase. The volume that is served permissively should be reflected in the proportion associated with the phase in which the permissive operation occurs. For example, consider an upstream cross street left-turn movement that serves 100 veh/h during the protected left-turn phase and 50 veh/h permissively during the cross street through phase. The upstream cross street right-turn movement serves 120 veh/h during the cross-street through phase and 80 veh/h as right-turn-on-red. All of the left- and right-turning vehicles are destined for the downstream through lane group, which has a volume of 1000 veh/h. Thus, the proportion of the 1000 veh/h from the upstream cross street left-turn phase is 0.10 (= 100/1000). The proportion of the 1000 veh/h from the upstream cross street through phase that serves the right-turn movement is 0.17 (= [120+50]/1000). If mid-segment entry contributes 0.05 to the 1000 veh/h, then the balance of 0.68 (= 1.0 - 0.10 - 0.17 - 0.05) represents the proportion of the through lane group volume originating from the upstream through movement phase.

**Volume Computations**

The procedure for estimating the arrival flow profile consists of completing the analysis steps outlined in this subsection and subsequent subsections.

1. Compute the volume flowing from movement \( u \) to lane group \( d \) (\( V_{u,d} \)). An initial step in the analysis is to decompose the lane group volume at the subject downstream intersection into the flow contribution from each of the upstream traffic movements. This decomposition is achieved by multiplying the lane group volume \( V_{\bullet,d} \) by the specified distribution \( P_{u,d} \). The equation describing this computation is:

\[
V_{u,d} = V_{\bullet,d} \cdot P_{u,d}
\]  

(12)

where,
- \( V_{u,d} \) = volume departing from movement \( u \) and destined for lane group \( d \), veh/h;
- \( P_{u,d} \) = proportion of volume from lane group \( d \) originating from the upstream movement \( u \);
- \( V_{\bullet,d} \) = volume for lane group \( d \), summed for all upstream movements \( u \), veh/h;
- \( d \) = subscript designating downstream exit lane group, 1 = left-turn lane group, 2 = through lane group, 3 = right-turn lane group, 4 = mid-segment exit; and
- \( u \) = subscript designating upstream entry traffic movement, 1 = left-turn from cross street, 2 = through from major street, 3 = right-turn from cross street, 4 = mid-segment entry.

If four lane groups and four traffic movements exist, then Equation 12 is used 16 times to estimate 16 volumes.

2. Compute the proportion of the upstream volume that contributes to the downstream lane group volume (\( P_{u,d}^{*} \)). This proportion is computed as:

\[
P_{u,d}^{*} = \frac{V_{u,d}}{\sum_{d=1}^{4} V_{u,d}}
\]  

(13)

C-12
where,
\[ P^{*}_{u,d} = \text{proportion of volume served by upstream movement } u \text{ that contributes to the volume of the downstream lane group } d. \]

If four lane groups and four upstream traffic movements exist, then Equation 13 can be used 16 times to estimate 16 proportions. However, if an arrival flow profile is desired for only the through lane group \( (d = 2) \), then Equation 13 will only need to be applied four times--once for each upstream movement \( u \).

**Segment Entry Flow Computations**

1. Compute the entry movement volume \( (V_{u,*}) \). This variable represents the volume for each of the traffic movements that enter the segment. It also includes the volume of traffic entering the segment via the mid-segment access points. This volume is computed as:
   \[ V_{u,*} = \sum_{d=1}^{4} V_{u,d} \quad (14) \]
   \[ V_{u,*} = \text{volume from upstream movement } u, \text{ summed for all downstream movements } d, \text{ veh/h.} \]

2. Compute the lane flow rate \( (q_u) \). This rate is computed for upstream traffic movement \( u \) as:
   \[ q_u = \frac{V_{u,*}}{3600 \ n_u} \quad (15) \]
   \[ q_u = \text{flow rate from upstream movement } u, \text{ veh/s/ln; and} \]
   \[ n_u = \text{number of lanes serving upstream movement } u. \]

3. Compute the effective green duration \( (g_u) \). This duration is computed for upstream traffic movement \( u \) as:
   \[ g_u = G_u + Y_u - t_{1u} - t_{2u} \quad (16) \]
   \[ g_u = \text{effective green duration for upstream movement } u, \text{ s;} \]
   \[ G_u = \text{green interval duration for upstream movement } u, \text{ s;} \]
   \[ Y_u = \text{yellow change plus red clearance interval duration for upstream movement } u, \text{ s;} \]
   \[ t_{1u} = \text{start-up lost time for upstream movement } u, \text{ s; and} \]
   \[ t_{2u} = \text{end lost time for upstream movement } u, \text{ s.} \]

The mid-segment entry movement can be assumed to flow continuously during the signal cycle. Also, if the upstream right-turn from the cross street is provided an exclusive right-turn lane, then it may also flow continuously throughout the cycle. Equation 16 and the remaining computations in this subsection (with one noted exception) are not used for “continuous” movements.
4. Compute saturation flow rate ($s_u$). This rate is computed for upstream traffic movement $u$ as:

$$s_u = \frac{s_{q|u}}{3600} \quad (17)$$

where,

$s_u =$ saturation flow rate for upstream movement $u$, veh/s/ln; and
$s_{q|u} =$ discharge flow rate for upstream movement $u$, veh/h/ln.

5. Compute volume-to-capacity ratio ($X_u$). This rate is computed for upstream traffic movement $u$ as:

$$X_u = \frac{q_u C}{s_u g_u} \quad (18)$$

where,

$X_u =$ volume-to-capacity ratio for upstream movement $u$; and
$C =$ cycle length, s.

If the volume-to-capacity ratio exceeds 1.0 for a movement, then the volume entering the segment for that movement will be set equal to its capacity. The additional vehicles are assumed to queue on the upstream movement approach.

6. Compute the number of time steps that flow at the saturation flow rate ($g_{q|u}'$). This number is computed for upstream traffic movement $u$ as:

$$g_{q|u}' = \text{Integer} \left( \frac{q_u (C - g_u)}{d_t (s_u - q_u)} \right) \leq \frac{g_u}{d_t} \quad (19)$$

where,

$g_{q|u}' =$ service time for queued vehicles in upstream movement $u$, steps;
$\text{Integer} [x] =$ round $x$ down to the nearest integer value (e.g., $\text{Integer} [8.9] = 8$); and
$\text{Round} [x] =$ round $x$ up or down to the nearest integer value (e.g., $\text{Round} [8.9] = 9$).

7. Compute the number of time steps that flow at the random arrival rate ($g_{r|u}'$). This number is computed for upstream traffic movement $u$ as:

$$g_{r|u}' = \text{Round} \left[ \frac{g_u}{d_t} - g_{q|u}' - 1.0 \right] \quad (20)$$

where,

$g_{r|u}' =$ service time for random arrivals in upstream movement $u$, steps; and
$\text{Round} [x] =$ round $x$ up or down to the nearest integer value (e.g., $\text{Round} [8.9] = 9$).

8. Compute the random arrival rate per time step ($q_u'$). This rate is computed for upstream traffic movement $u$ as:

$$q_u' = q_u d_t \quad (21)$$
where,
\[ q'_u = \text{arrival rate of upstream movement } u, \text{ veh/step/ln.} \]

This flow rate should be computed for all upstream entry traffic movements (including the middle-segment entry movement and any other movements that flow continuously throughout the cycle).

9. Compute the saturation flow rate per time step \( (s'_u) \). This rate is computed for upstream traffic movement \( u \) as:

\[ s'_u = s_u d_t \]  \hspace{1cm} (22)

where,
\[ s'_u = \text{saturation flow rate of upstream movement } u, \text{ veh/step/ln.} \]

10. Compute the flow rate for the step that follows the service time for queued vehicles \( (q'_{n|u}) \). This rate is computed for upstream traffic movement \( u \) as:

\[ q'_{n|u} = (q_u C) - (q'_{q|u} s'_u) - (s'_{u} q'_{u}) \leq s'_u \]  \hspace{1cm} (23)

where,
\[ q'_{n|u} = \text{intermediate step flow rate for upstream movement } u, \text{ veh/step/ln.} \]

This flow rate is assigned to the step that follows the service time for queued vehicles. It is an intermediate step and its flow rate is typically larger than the random arrival rate. By definition, this rate cannot exceed the saturation flow rate.

11. Identify the step associated with the start of green for upstream movement \( u \) \( (t'_{g|u}) \). This step is computed for upstream traffic movement \( u \) as:

\[ t'_{g|u} = \text{Round} \left[ \frac{t_{q|u} + t_{1|u}}{d_t} \right] \]  \hspace{1cm} (24)

where,
\[ t'_{g|u} = \text{start of effective green for upstream movement } u, \text{ steps.} \]

12. Identify the step associated with the end of the queue service period for upstream movement \( u \) \( (t'_{q|u}) \). This step is computed for upstream traffic movement \( u \) as:

\[ t'_{q|u} = t'_{g|u} + s'_{q|u} \]  \hspace{1cm} (25)

where,
\[ t'_{q|u} = \text{end of queue service period for upstream movement } u, \text{ steps.} \]

13. Identify the step associated with the end of effective green for upstream movement \( u \) \( (t'_{e|u}) \). This step is computed for upstream traffic movement \( u \) as:

\[ t'_{e|u} = t'_{q|u} + s'_{e|u} + 1.0 \]  \hspace{1cm} (26)
where,
\[ t'_{eu} = \text{end of effective green for upstream movement } u, \text{ steps}. \]

14. Compute the segment entry flow rate for time step \( i \) and upstream movement \( u \) (\( q'_{u,i} \)). Step \( i \) ranges from 0 to \( C/d_t \) in increments of 1.0. This flow rate is computed using the following rules:

- For entry by mid-segment turn or other continuous movements:
  \[ q'_{u,i} = q'u \]

- For all other upstream movements:
  - If \( t'_{q|u} < i < t'_{q|u} + 1.0 \) then \( q'_{u,i} = q_{n|u} \)
  - Otherwise, \( q'_{u,i} = 0.0 \)

where,
\[ q'_{u,i} = \text{entry flow rate for time step } i \text{ and upstream movement } u, \text{ veh/step/ln}. \]

The rules above are applied for each time step \( i \) in the signal cycle. There may be 30 to 60 time steps in the typical signal cycle, depending on the selected time step duration and the cycle length.

**Movement Arrival Flow Computations**

1. Compute the running time to the subject downstream intersection (\( T'_r \)). This time is computed as:

   \[ T'_r = \frac{D}{1.47 \cdot S_R \cdot d_t} \]  \hspace{1cm} (27)

   where,
   \[ T'_r = \text{average running time, steps}. \]

2. Compute the smoothing factor (\( F \)). This factor is computed as:

   \[ F = \frac{1}{1 + 0.138 \cdot T'_r + 0.315/d_t} \]  \hspace{1cm} (28)

   where,
   \[ F = \text{smoothing factor}. \]

   The constants in Equation 28 represent calibration coefficients based on field data. The steps undertaken to quantify these factors are described in the next part of the appendix.

3. Compute platoon arrival time (\( t' \)). This time is computed as:

   \[ t' = \text{Round} \left[ T'_r - \frac{1}{F} + 1 + 0.25 \right] \]  \hspace{1cm} (29)

   where,
   \[ t' = \text{platoon arrival time, steps}. \]
The constant “0.25” in Equation 29 represents a calibration coefficient based on field data. The steps undertaken to quantify this factor are described in the next part of the appendix.

4. Compute the downstream time step \( j \) for each upstream time step \( i \) \((j)\). This value is computed using the following rules:

- If \( i + t' < C/d_i \) then \( j = i + t' \)
- If \( i + t' \geq C/d_i \) then \( j = i + t' - C/d_i \)

The rules above are applied for each time step \( j \) in the signal cycle.

5. Compute the arrival flow rate for time step \( j \) and movement \( u \) \((q'_{a|u,j})\). This flow rate is computed as:

\[ q'_{a|u,j} = F \cdot q'_{a|t,i} + (1 - F) \cdot q'_{a|u,k} \]  \hspace{1cm} (30)

where,
- \( q'_{a|u,j} \) = arrival flow rate for time step \( j \) and movement \( u \), veh/step/ln; and
- \( q'_{a|u,k} \) = arrival flow rate for previous time step \( j-1 \) and movement \( u \), veh/step/ln.

The subscript \( k \) in the equation is subject to the following rules:

- If \( j > 0 \) then \( k = j - 1.0 \)
- If \( j = 0 \) then \( k = C/d_i - 1.0 \)

The equation and rules above are applied for each time step \( j \) in the signal cycle.

For the first time step \((i = 0)\), the flow rate from the previous time step \( q'_{a|u,k} \) should be estimated as 0.0 veh/step/ln. After all steps in the signal cycle have been evaluated, if the flow rate in the last step for a traffic movement is non-zero, then this flow rate should be used as an improved estimate of \( q'_{a|u,k} \) for the first time step and Equation 30 reapplied to each time step \( j \) for this movement. This process is iterative when the initial estimate of \( q'_{a|u,k} \) does not equal that found after all steps have been evaluated. Convergence is achieved when the absolute difference between the estimated and final flow rates is less than 0.005 veh/step/ln.

6. Compute the arrival flow profile for a given lane group in time step \( j \) \((Q'_{d,j})\). This flow rate is computed as:

\[ Q'_{d,j} = \sum_{u=1}^{4} q'_{a|u,j} \cdot n_u \cdot P_{u,d} \]  \hspace{1cm} (31)

where,
- \( Q'_{d,j} \) = arrival flow profile for lane group \( d \) in time step \( j \), veh/step.

The equation above is applied for each time step \( j \) in the signal cycle.
It is most likely that Equation 31 will be used to estimate the arrival flow for the through lane group, in which case, the subscript \(d\) will have the value 2.

**Proportion of Time Blocked**

The sequence of calculations for estimating the “proportion of time blocked” is described in this section. This proportion reflects the time that a dense platoon is present in an unsignalized intersection, relative to the signal cycle length. During this time period, the platoon is sufficiently dense as to preclude a minor movement driver from finding an acceptable gap. The blocked period is defined as having a flow rate that is associated with platoon headways that are too short to be entered (or crossed) by minor movements at an intersection. The maximum platoon headway for blocking to occur is estimated as 3.6 s/veh, although other rates can be rationalized based on the type of maneuver and the number of lanes crossed.

The following discussion describes the steps for computing the proportion of time blocked. These steps were developed in a generalized manner such that they can be applied to any downstream lane group. If they are applied to the through lane group, then the subscript \(d\) should have the value 2. The through lane group is typically the only group that presents significant conflict for the minor movements.

1. Compute the minimum flow rate above which headways are too short to be entered (or crossed) by minor movements \(q'_{b}\). This time is computed as:

\[
q'_{b} = \frac{d_{i}}{t_{b}}
\]

where,
\(q'_{b}\) = minimum blocking flow rate, veh/step.

2. Compute the downstream time step \(j\) for each upstream time step \(i\) \((j)\). The rules that apply in this computation are described in the Step 4 of the subsection titled Movement Arrival Flow Computations. These rules are applied for each time step \(j\) in the signal cycle.

3. Compute the conflicting arrival flow rate \(Q'_{cij}\). This rate is computed as:

\[
Q'_{cij} = \sum_{d} Q'_{dji}
\]

where,
\(Q'_{cij}\) = conflicting arrival flow rate, veh/step.

The conflicting arrival flow rate represents the sum of all lane groups in the approaching traffic stream that conflict with minor movement flows at the unsignalized intersection. At most, the sum would include the arrival flow profile for the left-turn, through, and right-turn lane groups (i.e., \(d = 1, 2,\) and 3). Typically, the through lane group (i.e., \(d = 2\)) is the only one that presents a significant conflict, and the summation can be simplified to include only this lane group.
4. Establish the blocked condition flag for time step \( j \) \( (I_{bj}) \). This value is computed using the following rule:

- If \( Q'_{eij} > q'_b \) then \( I_{bj} = 1 \)
- Otherwise, \( I_{bj} = 0 \)

The rule above is applied for each time step \( j \) in the signal cycle.

5. Compute the total time in the cycle that is blocked by a platoon \( (t'_p) \). For the left-turn movement from the major street or the cross street right-turn movement, this time is computed as:

\[
t'_p = \sum_{j=0}^{m} I_{bj} \tag{34}
\]

where,

- \( t'_p \) = blocked period duration, steps;
- \( m \) = number of steps in signal cycle (= \( C/d \)).

For the cross street through movement or the cross street left-turn movement, additional calculations are required because these movements cross the traffic stream arriving at the intersection from both travel directions. To estimate the blocked period duration for these two cross street movements, the arrival flow profile must also be computed for the opposing travel direction. It would then be used to compute the blocked condition flag for each time step in the opposing direction \( I_{bolj} \). The blocked condition flags from both directions of travel would then be combined in the following equation to estimate the blocked period duration:

\[
t'_p = \sum_{j=0}^{m} \left[ 1.0 - (1.0 - I_{bj}) \left( 1.0 - I_{bolj} \right) \right] \tag{35}
\]

where,

- \( I_{bolj} \) = blocked condition flag for the opposing travel direction.

This equation requires the cycle length for the upstream signalized intersection in both travel directions to be divided into the same number of steps \( m \).

6. Compute the proportion of time blocked \( (p) \). This value is computed using the following equation.

\[
p = \frac{t'_p \cdot d_t}{C} \tag{36}
\]

where,

- \( p \) = proportion of time blocked.

As the preceding calculation steps indicate, the percent time blocked is not influenced by signal offset.
Proportion of Vehicles Arriving on Green

The sequence of calculations for estimating the “proportion of vehicles arriving on green” is described in this section. This proportion reflects the number of vehicles that arrive at the downstream signalized intersection during the effective green period of a specified signal phase. This proportion is most commonly computed for the through phase; however, it could also be computed for the left-turn phase.

The following discussion describes the steps for computing the proportion of vehicles arriving on green. These steps were developed in a generalized manner such that they can be applied to any downstream lane group. If they are applied to the signal phase that serves the through lane group, then the subscript \(d\) should have the value 2. This lane group is typically the lane group that is progressed when part of a signal system.

1. Compute the start of the effective green period for the subject lane group \(t_{g|d}\). This time is computed as:

\[
t_{g|d} = \text{Mod}\left[O_r + \sum (G_j + Y_j) + t_{1|d}\right]
\]

where,
- \(t_{g|d}\) = start of the effective green period for lane group \(d\), s;
- \(t_{1|d}\) = start-up lost time for lane group \(d\), s; and
- \(\text{Mod}\) \([x]\) = remainder after the largest integer multiple of the cycle length is removed from \(x\) (e.g., if the cycle length is 110 s, \(\text{Mod}\) \([222]\) = 222 - 2 \times 110 = 2).

The summation term in the equation represents the sum of all downstream phases that occur prior to the subject phase, starting with the downstream major-street through movement. If the subject lane group is the major-street through movement, then the summation term is equal to 0.0.

2. Compute the end of the effective green period for the subject lane group \(t_{e|d}\). This time is computed as:

\[
t_{e|d} = \text{Mod}\left[t_{g|d} + G_d + Y_d - i_{1d} - t_{2|d}\right]
\]

where,
- \(t_{e|d}\) = end of the effective green period for lane group \(d\), s.

3. Compute the beginning of interval A for the subject lane group \(t'_{ab|d}\). This value is computed as:

\[
t'_{ab|d} = \text{Round}\left[\frac{t_{g|d}}{d_i}\right]
\]

where,
- \(t'_{ab|d}\) = beginning of interval A for lane group \(d\), steps.

Two intervals are defined in this and subsequent steps. These intervals identify the range of steps that define the effective green period for the subject lane group.
4. Compute the end of interval A for the subject lane group \( t'_{ae|d} \). This value is computed using the following rule:
   
   - If \( t_{el|d} < t_{gl|d} \) then \( t'_{ae|d} = \text{Round}[C/d] \)
   - Otherwise, \( t'_{ae|d} = \text{Round}[t_{el|d}/d] \)
   
   where,
   \( t'_{ae|d} = \text{end of interval A for lane group } d, \text{ steps.} \)

5. Compute the beginning of interval B for the subject lane group \( t'_{bb|d} \). This value is computed using the following rules:
   
   - If \( t_{el|d} < t_{gl|d} \) then \( t'_{bb|d} = 0.0 \)
   - Otherwise, \( t'_{bb|d} = t'_{ab|d} \)
   
   where,
   \( t'_{bb|d} = \text{beginning of interval B for lane group } d, \text{ steps.} \)

6. Compute the end of interval B for the subject lane group \( t'_{be|d} \). This value is computed as:
   
   \[
   t'_{be|d} = \text{Round}\left[\frac{t_{el|d}}{d}\right]
   \]  
   (40)

   where,
   \( t'_{be|d} = \text{end of interval B for lane group } d, \text{ steps.} \)

7. Compute the downstream time step \( j \) for each upstream time step \( i \) \((j)\). The rules that apply in this computation are described in the Step 4 of the subsection titled Movement Arrival Flow Computations.

8. Establish the effective green flag for time step \( j \) \((I_{gl|d,j})\). This value is computed using the following rule:
   
   - If \( t_{ab|d} \leq j < t'_{ae|d} \) or \( t'_{bb|d} \leq j < t'_{be|d} \) then \( I_{gl|d,j} = 1 \)
   - Otherwise, \( I_{gl|d,j} = 0 \)

   The rule above is applied for each time step \( j \) in the signal cycle.

9. Compute the combined arrival flow during effective green for time step \( j \) \((Q'_{gl|d,j})\). This flow rate is computed as:
   
   \[
   Q'_{gl|d,j} = Q'_{d|j} I_{gl|d,j}
   \]  
   (41)

   where,
   \( Q'_{gl|d,j} = \text{combined arrival flow during effective green for time step } j, \text{ veh/step.} \)

The equation and rules above are applied for each time step \( j \) in the signal cycle.
10. Compute the total arrival volume during the effective green period ($V_{g|d}$). This volume is computed as:

$$V_{g|d} = \sum_{j=0}^{m} Q'_{g|d,j}$$  \hspace{1cm} (42)

where,

$V_{g|d} =$ arrival volume during effective green, veh/cycle;
$m =$ number of steps in signal cycle ($= C/d$).

11. Compute the proportion of vehicles arriving during the effective green period for the subject lane group ($P_d$). This proportion is computed using the following equation.

$$P_d = \frac{3600 \cdot V_{g|d}}{V_{s|d} \cdot C}$$ \hspace{1cm} (43)

where,

$P_d =$ proportion of vehicles arriving on green for lane group $d$.

12. Compute the platoon ratio for the subject lane group ($R_{p|d}$). This ratio is computed as:

$$R_{p|d} = \frac{3600 \cdot V_{g|d}}{V_{s|d} \cdot g_d}$$ \hspace{1cm} (44)

where,

$R_{p|d} =$ platoon ratio for lane group $d$; and
$g_d =$ effective green duration for lane group $d$ ($= G_d + Y_d - t_{1d} - t_{2d}$), s.

13. Compute the progression adjustment factor for the subject lane group ($PF_d$). This ratio is computed as:

$$PF_d = \frac{1 - P_d}{1 - \frac{g_d}{C}}$$ \hspace{1cm} (45)

where,

$PF_d =$ platoon ratio for lane group $d$.

**SENSITIVITY ANALYSIS**

This part of the appendix examines the sensitivity of the arrival flow profile prediction procedure described in the previous part. The examination considers the effect of traffic volume, volume distribution, signal offset, and travel distance on the arrival flow profile. This effect is quantified in terms of: (1) the proportion of time an intersection is blocked by a platoon, and (2) the proportion of vehicles arriving during the effective green period of the phase serving the through lane group.

**Proportion of Time Blocked**

The analysis of “proportion time blocked” focused on the evaluation of two volume scenarios. Both scenarios were based on a segment exit flow of 1000 veh/h, a left-turn lane group volume of 100 veh/h, a through lane group that includes right-turn vehicles in a shared lane, a
running speed of 45 mph, and a cycle length of 120 s. One scenario had 600 veh/h exiting the
downstream intersection via the through lane group and 300 veh/h exiting via mid-segment access
points. The other scenario had 850 veh/h exiting via the through lane group and 50 veh/h exiting
via mid-segment access points. The findings from this analysis are shown in Figure C-6.

Two trend lines are shown in Figure C-6. The upper trend line represents the scenario with
high through lane group volume and low access point volume. This combination creates a lengthy,
dense platoon at the start of the segment, followed by a fairly small volume for the remainder of
the signal cycle. The amount of dispersion that occurs with increasing distance is not sufficient to create
gaps in the platoon through which minor movements can enter (or cross). As a result, the dispersion
that does occur only increases the length of the platoon and, thereby, increases the proportion of time
blocked.

The lower trend line in Figure C-6 represents the scenario with moderate through volume and
high access point volume. This combination creates a moderate sized platoon at the start of the
segment, followed by a moderate volume for the remainder of the signal cycle. The significant
access point volume tends to “decay” the platoon as vehicles depart at various intermediate access
points. There is also some dispersion that occurs as platooned drivers tend to increase their headway
as they travel down the street. The combined effect of platoon dispersion and decay creates gaps in
the platoon through which minor movements can enter. As a result, the proportion of time blocked
decreases with increasing travel distance.

Examination of the two trend lines in Figure C-6 suggests that there is likely to be some
volume combinations that result in a horizontal trend line. A horizontal trend line would indicate
that the proportion of time blocked does not change with distance to the upstream signalized
intersection.
Proportion of Arrivals on Green

The analysis of “proportion of vehicles arriving on green” focused on an evaluation of the same two volume scenarios as described in the previous section. In addition, an access point density of 40 a.p./mi was assumed. For the scenario with moderate through volume and high access point volume, the delay resulting from access point activity was assumed to be 0.5 s/veh. The delay for the high through/low access-point volume was assumed to be negligible. The findings from this analysis are shown in Figure C-7.

Figure C-7. Effect of volume and offset on proportion of arrivals on green.

Two trend lines are shown in each of Figures C-7a and C-7b. One trend line coincides with a segment length of 1320 ft and the other coincides with a length of 5280 ft. Both trend lines exhibit a sinusoidal variation indicating the effect of signal offset on progression quality. In Figure C-7a, the 1320 ft segment yields the maximum proportion of arrivals on green of 0.72 at an offset of 20 s. This offset equates to the segment running time of 20 s (=1320/[1.47 ×45]). For the 5280 ft segment, the maximum proportion of 0.60 occurs at an offset of 75 s. Other offsets reduce the number of vehicles that arrive during the effective green period.

A comparison of the two trend lines in Figure C-7a indicates the maximum proportion for the 1320 ft segment is larger than that for the 5280 ft segment. Similarly, the minimum proportion for the 1320 ft segment is smaller than that for the 5280 ft segment. In other words, the peaks and valleys of the 5280 ft trend line are “moderated,” relative to those of the 1320 ft trend line. This moderation is due to the dispersion of the platoon as it travels down the segment. Dispersion occurs when platoon drivers increase their headway with increasing running time.

Figure C-7b illustrates similar trends as observed in Figure C-7a. The peaks and valleys of both trend lines are significantly moderated, relative to those shown in Figure C-7a. The moderation is due largely to the fact that many of the vehicles in the platoon entering the segment ultimately exit at a mid-segment access point. The moderation is also due partly to the platoon dispersion effect.
In Figure C-7b, the maximum proportion of arrivals of 0.37 occurs at an offset of 95 s for the 5280 ft segment. This offset compares to the offset of 75 for the 5280 ft segment in Figure C-7a. The additional 20 s (= 95 - 75) is an indirect result of the “friction” caused by the high access point volume represented in Figure C-7b. As noted previously, this volume was sufficiently high as to pose a 0.5 s/veh delay at each access point. When summed for all 40 access points along the segment, this small amount of delay increased segment running time by 20 s (= 0.5 × 40) which resulted in the noted increase in optimum offset.

This sensitivity analysis highlights the two primary causes for dispersed flow patterns in an arrival flow profile. They are:

- Vehicles departing the platoon at mid-segment access points decay the platoon structure by leaving a void between each departing vehicle and the vehicle just ahead of, and behind, it.
- Drivers in a platoon naturally seek to increase their headway (i.e., spread out) as they travel down the street. The amount of platoon dispersion increases with an increase in running time.

The smoothing factor $F$ used in the platoon dispersion model relates running time to platoon dispersion. The effect on platoon dispersion of mid-segment activity or design elements that increase running time is modeled through this factor. On the other hand, the distribution of upstream traffic movements to the downstream lane group accounts for platoon structure decay. Vehicles exiting the platoon at mid-segment access points are a common source of this decay.

The sensitivity analysis indicates that platoon decay tends to have a more significant impact on the arrival flow profile than platoon dispersion. Accurate modeling of platoon dispersion (in the absence of decay effects) requires minimizing the effects of platoon decay by selecting sites with negligible access point activity, or by collecting data flow profile data for only those vehicles that traverse the entire segment. If these types of measures are not taken and the platoon dispersion model is calibrated with traffic streams that include access point traffic, the calibrated smoothing factor will reflect both dispersion and decay effects and its value will be larger than that if it represented only dispersion.

SELECTED MODEL CALIBRATION

This part of the appendix summarizes the findings from the calibration of the platoon dispersion model. The summary is provided in two sections. The first section describes the study sites and summarizes the data collected at each site. The second section summarizes the findings from the model calibration.

Study Site Description and Database Summary

Calibration data were collected at ten urban study sites. Each site represents one direction of travel on one arterial street segment. The sites were selected such that they collectively represented a range of geographic locations, urban street classes, segment lengths, speed limits, and traffic flow rates. Table C-1 summarizes the characteristics of the ten study sites. A more complete description of each site was provided previously in Appendix B.
### TABLE C-1 Study segment summary table

<table>
<thead>
<tr>
<th>Site</th>
<th>Corridor</th>
<th>Location</th>
<th>Street Class</th>
<th>Segment Length, ft</th>
<th>Speed Limit, mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Campbell Road</td>
<td>Richardson, Texas</td>
<td>II</td>
<td>1102</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>H. Mitchell Parkway</td>
<td>College Station, Texas</td>
<td>II</td>
<td>1506</td>
<td>45</td>
</tr>
<tr>
<td>3</td>
<td>H. Mitchell Parkway</td>
<td>College Station, Texas</td>
<td>I</td>
<td>2330</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>Valley View Lane</td>
<td>Farmers Branch, Texas</td>
<td>III</td>
<td>2611</td>
<td>35</td>
</tr>
<tr>
<td>5</td>
<td>Wellborn Road</td>
<td>College Station, Texas</td>
<td>II</td>
<td>3704</td>
<td>45</td>
</tr>
<tr>
<td>6</td>
<td>S.R. 823 (Flamingo Rd.)</td>
<td>Fort Lauderdale, Florida</td>
<td>I</td>
<td>2650</td>
<td>50</td>
</tr>
<tr>
<td>7</td>
<td>S.R. 842 (Broward Blvd.)</td>
<td>Fort Lauderdale, Florida</td>
<td>III</td>
<td>996</td>
<td>35</td>
</tr>
<tr>
<td>8</td>
<td>S.E. Powell Boulevard</td>
<td>Portland, Oregon</td>
<td>II</td>
<td>1405</td>
<td>35</td>
</tr>
<tr>
<td>9</td>
<td>S.E. McLoughlin Blvd.</td>
<td>Portland, Oregon</td>
<td>I</td>
<td>2123</td>
<td>45</td>
</tr>
<tr>
<td>10</td>
<td>S.W. Barbur Boulevard</td>
<td>Portland, Oregon</td>
<td>II</td>
<td>2937</td>
<td>35</td>
</tr>
</tbody>
</table>

Note:
1 - Street class designations are based on the description provided in Exhibits 10-3 and 10-4 of the *Highway Capacity Manual* (1).

The videotapes recorded during each study were replayed for the purpose of manually extracting the data needed for model calibration. Of the three hours of video tape recorded at each site, 45 minutes of data were extracted for each site. Specifically, one 15-minute sample of data was extracted from each one-hour videotape for a total of 30, 15-minute samples from the 10 sites.

The data collected for platoon dispersion model calibration consist of: (1) the time each vehicle crossed a reference mark on the pavement, (2) its manner of entry to the segment, and (3) the vehicle’s classification and color. In general, the “upstream” reference mark was located 400 to 600 ft downstream of the upstream signalized intersection. The “mid-segment” reference mark was located at about the middle of the segment. The “downstream” reference mark was located about 400 to 600 ft upstream of the downstream signalized intersection. A downstream reference mark was established only at the six longer sites (i.e., sites 3, 4, 5, 6, 9, and 10).

For the six longer sites, vehicles were matched for the partial segment between the upstream and downstream marks. For the four shorter sites, vehicles were matched only for the partial segment between the upstream and mid-segment marks. The running time for each matched vehicle was added to obtain a total running time for each cycle and site. This total was divided into the partial segment length to estimate the cycle running speed. This method of computing running speed yielded the desired space-mean speed estimate. Any vehicle that stopped mid-segment was excluded from this database.

The data reduction statistics are summarized in Table C-2 for the longest partial segment established at each site. As indicated in the last row of this table, a total of 5883 “through” vehicles were tracked along the partial segments during the thirty, 15-minute time periods. For dispersion model calibration, a through vehicle was defined as a vehicle that entered the segment as a through vehicle and crossed the downstream reference mark as a through vehicle. Vehicles that entered or exited at an access point were excluded from this analysis. This focus on through vehicles was
intended to ensure that the model was calibrated to predict platoon dispersion, as opposed to platoon decay.

**TABLE C-2 Data reduction statistics for partial segment study sites**

<table>
<thead>
<tr>
<th>Site</th>
<th>Corridor</th>
<th>From</th>
<th>To</th>
<th>Through Vehicles</th>
<th>Running Speed, mph</th>
<th>Dispersion Coeff. α</th>
<th>Arrival Time Coeff. β</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Campbell Road</td>
<td>N. Collins Boulevard</td>
<td>Canyon Creek Drive</td>
<td>778</td>
<td>37.5</td>
<td>0.16</td>
<td>0.91</td>
</tr>
<tr>
<td>2</td>
<td>Harvey Mitchell Parkway</td>
<td>Longmire Drive</td>
<td>Southwood Drive</td>
<td>457</td>
<td>38.3</td>
<td>0.13</td>
<td>0.95</td>
</tr>
<tr>
<td>3</td>
<td>Harvey Mitchell Parkway</td>
<td>Welsh Avenue Drive</td>
<td>Rio Grande Boulevard</td>
<td>307</td>
<td>45.1</td>
<td>0.16</td>
<td>0.86</td>
</tr>
<tr>
<td>4</td>
<td>Valley View Lane</td>
<td>Josey Lane</td>
<td>Webb Chapel Road</td>
<td>278</td>
<td>38.6</td>
<td>0.15</td>
<td>0.88</td>
</tr>
<tr>
<td>5</td>
<td>Wellborn Road</td>
<td>George Bush Drive</td>
<td>Holleman Drive</td>
<td>353</td>
<td>42.7</td>
<td>0.26</td>
<td>0.85</td>
</tr>
<tr>
<td>6</td>
<td>S.R. 823 (Flamingo Road)</td>
<td>S.W. Taft Street</td>
<td>S.W. Johnson St.</td>
<td>987</td>
<td>47.6</td>
<td>0.16</td>
<td>0.90</td>
</tr>
<tr>
<td>7</td>
<td>S.R. 842 (Broward Boulevard)</td>
<td>N.E. 3rd Avenue</td>
<td>Andrews Avenue</td>
<td>459</td>
<td>32.0</td>
<td>0.23</td>
<td>0.89</td>
</tr>
<tr>
<td>8</td>
<td>S.E. Powell Boulevard</td>
<td>S.E. 21st Avenue</td>
<td>S.E. 26th Avenue</td>
<td>1089</td>
<td>29.1</td>
<td>0.24</td>
<td>0.85</td>
</tr>
<tr>
<td>9</td>
<td>S.E. McLoughlin Boulevard</td>
<td>S.E. 17th Avenue</td>
<td>S.E. Holgate Boulevard</td>
<td>851</td>
<td>44.4</td>
<td>0.13</td>
<td>0.93</td>
</tr>
<tr>
<td>10</td>
<td>S.W. Barbur Boulevard</td>
<td>S.W. 19th Avenue</td>
<td>S.W. Bertha Boulevard</td>
<td>324</td>
<td>40.6</td>
<td>0.36</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td><strong>Total:</strong></td>
<td></td>
<td></td>
<td><strong>5883</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note:**
1 - Vehicles entered the segment as a through movement at the upstream signal and crossed the downstream reference mark as a through movement. Total for three, 15-minute periods.

The last two columns of Table C-2 provide estimates of the platoon dispersion coefficient $\alpha$ and the platoon arrival time coefficient $\beta$. The dispersion coefficient for the collective set of sites ranged from 0.13 to 0.36, with an average of 0.20. This range falls at the lower end of the range reported by Axhausen and Korling (8) (i.e., 0.10 to 0.63) and is much lower than the value of 0.50 recommended by Robertson (2).

It is likely that the studies reviewed by Axhausen and Korling (8) did not expend the additional resources to track each vehicle through the study segment. Rather, it is likely that they were based only on the measurement of flow profiles at two reference marks. If a flow profile is simply measured at each of two marks, and the individual vehicles are not tracked through the segment, then it is likely that the profiles thus obtained will include both through and access point vehicles. If so, the calibration coefficients would include the effect of both platoon dispersion and
platoon decay, and consequentially, the $\alpha$ obtained would be larger than an $\alpha$ based solely on dispersion. This trend is evident when comparing the range of $\alpha$ reported by Axhausen and Korling (8) with that found in Table C-2 (which is based on the dispersion of only through vehicles).

The last column of Table C-2 indicates that the platoon arrival time coefficient $\beta$ ranges from 0.84 to 0.95, with an average of 0.89. This range is at the upper end of the range reported by Axhausen and Korling (8) (i.e., 0.59 to 0.99) and includes the value of 0.80 recommended by Robertson (2).

Model Calibration

This section describes the findings from the model calibration process. The calibration process consisted of three steps. Initially, a flow profile was computed for the beginning and the ending reference marks associated with each partial segment for each 15 minute analysis period. A time step duration of 2 s was specified and the time steps were numbered for each 2 s interval. The first time step at each mark was defined as the step that was coincident with the start of through phase green at the upstream intersection. The count of through vehicles crossing each mark in each time step was computed for each cycle. This count was then summed for all signal cycles during a 15 minute analysis period. The sum was used to compute a step flow rate in units of “vehicles per second per lane.” In this manner, the six longer sites had two pairs of flow profiles computed, one pair for each partial segment. The four shorter sites had only one pair of flow profiles computed for them. Thus, a total of 16 flow profile pairs were computed for each 15 minute analysis period, for a total of 48 pairs for the three hours of study at each site.

Figure C-8 illustrates a flow profile pair for Site 9 (using a dashed trend line). Each data point represents the average of the observed through vehicle count in a given time step for the 15 to 20 cycles observed at this site during the three, 15 minute analysis periods.

\begin{enumerate}
  \item \textit{a. Beginning of Partial Segment.}
  \item \textit{b. End of Partial Segment.}
\end{enumerate}

\textit{Figure C-8. Comparison of measured and predicted flow profiles.}
As a second step in the calibration process, the SAS nonlinear regression procedure (NLIN) was used to automate an iterative search for the optimal fit of Equations 1 and 2 combined to each of the 48 flow profiles. The search objective was to minimize the sum of the squared error between the predicted flow profile and the actual flow profile for each time step. The model fit to the data for Site 9 is shown in Figure C-8b. The quality of this fit is shown in Figure C-9.

![Figure C-9. Comparison of predicted and measured flow profiles at one site.](image)

Separate regression analyses were conducted for each of the 16 site and segment pair combinations. Each regression analysis yielded a best estimate of the smoothing factor $F$ and the platoon arrival time $t'$ for a site and segment pair. A total of 16 estimates of $F$ and $t'$ were obtained in this manner. The process was repeated for a time step of 1 s and again for a time step of 3 s to collectively yield 48 estimates of $F$ and $t'$ (one data set based on 1 s time step, a second on 2 s time step, and a third on 3 s time step).

As a third step in the calibration process, the 48 estimates of $F$ and $t'$ were used to calibrate separate equations for predicting $F$ and $t'$. The findings from this calibration process are described separately for the two equations in the following subsections. First, the findings from the smoothing factor equation calibration are described. Then, the findings from the platoon arrival time equation calibration are described.

**Smoothing Factor Equation**

The 48 estimates of the smoothing factor $F$ represented the dependent variable for equation calibration. The analysis initially focused on an equation that included the effect of average running time, step duration, and volume-to-saturation flow rate ratio. The running time of each vehicle that crossed both segment reference marks, regardless of whether they entered the segment as a through vehicle, was used to estimate the average running time.
Preliminary regression analyses indicated that the volume-to-saturation flow rate ratio was not correlated with the best estimates of $F$ or $t'$. The best-fit equation form was based on Equation 6, but modified to include regression coefficients for running time and time step duration. The resulting regression model is:

$$F = \frac{1}{1 + b_0 \frac{T_r}{d_i} + b_1/d_i}$$

(46)

where,

- $T_r =$ average running time, $s$;
- $d_i =$ time step duration, $s/\text{step}$; and;
- $b_i =$ calibration coefficient ($i = 0, 1$).

The nonlinear regression procedure (NLIN) provided in the SAS system was used to estimate the best-fit regression coefficients. This procedure uses an iterative search method to adjust model coefficients and, thereby, optimize the fit of non-linear regression models. Model fit was based on minimizing the sum of the squared error. Nonlinear regression was used because Equation 46 has a nonlinear relationship between the independent and dependent variables. To insure unbiased coefficients, weighted regression was used where the weight of each $F$ observation was set equal to the number of through vehicles observed at the corresponding site.

The statistics related to the calibrated model are shown in Table C-3. The calibrated coefficient values shown in the bottom half of the table can be used in Equation 46 to estimate the smoothing factor as a function of running time and time step duration. The $R^2$ for the model is 0.81, indicating that the model explains 81 percent of the variability in the data. The standard deviation of the model estimate is 0.080.

### TABLE C-3 Smoothing factor model statistics

<table>
<thead>
<tr>
<th>Model Statistics</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R^2$</td>
<td>0.81</td>
</tr>
<tr>
<td>Observations $n_0$</td>
<td>48 ($= 16$ partial segments $\times 3$ time step increments)</td>
</tr>
<tr>
<td>Standard Deviation $s$</td>
<td>±0.080</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Range of Model Variables</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable</td>
<td>Variable Name</td>
</tr>
<tr>
<td>----------</td>
<td>---------------</td>
</tr>
<tr>
<td>$T_r$</td>
<td>Running time</td>
</tr>
<tr>
<td>$d_i$</td>
<td>Time step duration</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Calibrated Coefficient Values</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable</td>
<td>Definition</td>
</tr>
<tr>
<td>---------</td>
<td>------------</td>
</tr>
<tr>
<td>$b_0$</td>
<td>Effect of running time</td>
</tr>
<tr>
<td>$b_1$</td>
<td>Effect of time step duration</td>
</tr>
</tbody>
</table>

One means of assessing the model’s fit is through a graphical comparison of the observed and predicted smoothing factors. This comparison is provided in Figure C-10. The trend line in this figure does not represent the line of best fit; rather, it is a “$y=x$” line. The data would lie on this line.
if the model predictions exactly equaled the observed data. The clustering of the data around this line indicates that the model is able to predict the smoothing factor without bias.

Figure C-10. Comparison of measured and predicted smoothing factors.

**Platoon Arrival Time Equation**

The 48 estimates of platoon arrival time $t'$ represented the dependent variable for equation calibration. Preliminary regression analyses indicated that the best-fit equation form was based on Equation 4, but modified to include an additional regression coefficient. The resulting regression model is:

$$ t' = \frac{T_r}{d_t} - \frac{1}{F} + c_0 $$

(47)

where,

- $t'$ = platoon arrival time, steps;
- $F$ = predicted smoothing factor from Equation 46; and;
- $c_0$ = calibration coefficient.

The linear regression procedure provided in the SAS system was used to estimate the best-fit regression coefficients. This procedure computes model coefficients that minimize the sum of the squared error. It is appropriate because Equation 47 represents a linear model form. To insure unbiased coefficients, weighted regression was used where the weight of each $t'$ observation was set equal to the number of through vehicles observed at the corresponding site.

The statistics related to the calibrated model are shown in Table C-4. The calibrated coefficient value shown in the bottom half of the table can be used in Equation 47 to estimate the platoon arrival time as a function of running time, time step duration, and smoothing factor. The $R^2$ for the model is 0.99, indicating that the model explains 99 percent of the variability in the data. The standard deviation of the model estimate is 0.37 steps.
### TABLE C-4  Platoon arrival time model statistics

<table>
<thead>
<tr>
<th>Model Statistics</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R^2$:</td>
<td>0.99</td>
</tr>
<tr>
<td>Observations $n$:</td>
<td>48 (= 16 partial segments × 3 time step increments)</td>
</tr>
<tr>
<td>Standard Deviation $s$:</td>
<td>±0.37 steps</td>
</tr>
</tbody>
</table>

#### Range of Model Variables

<table>
<thead>
<tr>
<th>Variable</th>
<th>Variable Name</th>
<th>Units</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_r$</td>
<td>Running time</td>
<td>s</td>
<td>5</td>
<td>42</td>
</tr>
<tr>
<td>$d_t$</td>
<td>Time step duration</td>
<td>s/step</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>$F$</td>
<td>Predicted smoothing factor</td>
<td>--</td>
<td>0.14</td>
<td>0.76</td>
</tr>
</tbody>
</table>

#### Calibrated Coefficient Values

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
<th>Value</th>
<th>Std. Dev.</th>
<th>t-statistic</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_0$</td>
<td>Intercept</td>
<td>0.254</td>
<td>0.088</td>
<td>2.9</td>
</tr>
</tbody>
</table>

A graphical comparison of the observed and predicted arrival times is provided in Figure C-11. The trend line in this figure does not represent the line of best fit; rather, it is a “$y = x$” line. The data would lie on this line if the model predictions exactly equaled the observed data. The clustering of the data around this line indicates that the model is able to predict arrival time without bias.

![Figure C-11. Comparison of measured and predicted platoon arrival time.](image)

#### REFERENCES


APPENDIX D

Procedure for Estimating Coordinated-Actuated Phase Duration
APPENDIX D

PROCEDURE FOR ESTIMATING
COORDINATED-ACTUATED PHASE DURATION

INTRODUCTION

This appendix describes a procedure for estimating the duration of each signal phase at an intersection that is located within a coordinated-actuated signal system (hereafter, referred to as the “phase duration procedure”). Actuated phase duration varies each cycle in response to random variations in traffic flow rate, and as constrained by the phases’ minimum and maximum green settings. In recognition of this variability, the procedure described here is developed to predict the long-run average phase duration for steady flow conditions. This average duration can be used with the procedure described in Chapter 16 of the *Highway Capacity Manual (HCM)* (1) to estimate the delay incurred by a traffic movement. It can also be used to estimate the number of stops incurred by this movement using the procedure described in Appendix E.

This appendix consists of four parts. The first part provides a review of actuated signal controller operation and a description of relevant models for estimating actuated phase duration. The second part describes the phase duration procedure. The third part describes the findings from a sensitivity analysis using the procedure. The last part summarizes the findings from the model calibration activity.

BACKGROUND

This part of the appendix describes background information related to the topic of actuated control and average phase duration. The first section provides a brief review of actuated control. The second section provides an overview of the procedure described in Chapter 16, Appendix B of the *HCM* for estimating the duration of actuated phases. The third section introduces “maximum allowable headway” as a characteristic for describing the relationship between detection design and the associated signal phase duration. The fourth section describes a procedure for estimating actuated phase duration.

Actuated Control

This section provides an overview of actuated signal control. Its purpose is primarily to supplement the discussion in Chapter 16, Appendix B of the *HCM* by providing some basic concepts that are inherent to understanding the phase duration procedure described in the next part of this appendix.

*Intersection Traffic Movements*

Figure D-1 illustrates typical vehicle and pedestrian traffic movements at a four-leg intersection. Three vehicular traffic movements and one pedestrian traffic movement are shown for
each intersection approach. Each movement is assigned a unique number, or a number and letter combination. The letter “R” denotes a right-turn movement. The letter “P” denotes a pedestrian movement.

**Figure D-1. Intersection traffic movements and numbering scheme.**

Intersection traffic movements are assigned the right-of-way by the signal controller in real time. Each movement is assigned to one or more signal phases. A signal phase is defined as the green, yellow change, and red clearance intervals in a cycle assigned to a specified movement (or movements) of traffic (2). The engineer’s assignment of movements to phases will vary in practice, depending on the desired phase sequence and the movements that are present at the intersection.

**Signal Phase Sequence**

Modern actuated controllers implement signal phasing using a dual-ring structure that allows for the concurrent presentation of a green indication to two phases. Each phase serves one or more movements that do not conflict with each other or those of the concurrent phase. Early controllers used a single-ring structure where all conflicting movements were assigned to a common phase and its duration was dictated by the one movement needing the most time. Of the two structures, the dual-ring structure is more efficient because it allows the controller to adapt phase duration and sequence to the needs of the individual movements. The dual-ring structure is typically used with eight phases; however, more phases are available for complex signal phasing. The eight-phase dual-ring structure is shown in Figure D-2. The symbol “Φ” shown in this figure represents the word “phase” and the number following the symbol represents the phase number.

Shown in Figure D-2 are the traffic movements sometimes assigned to each of the eight phases. These assignments are illustrative, but they are not uncommon. Each left-turn movement is assigned to an exclusive phase. During this phase, the left-turn movement is “protected” such that
it receives a green arrow indication. Each through, right turn, and pedestrian movement combination is also assigned to an exclusive phase. The dashed arrows indicate turn movements that are served in a “permissive” manner such that the turn can be completed only after yielding the right-of-way to conflicting protected movements.

![Dual-ring structure with illustrative movement assignments.](image)

Two rings and two barriers are identified in Figure D-2. A ring consists of two or more sequentially timed conflicting phases. Ring 1 consists of phases 1, 2, 3, and 4. Ring 2 consists of phases 5, 6, 7, and 8. A barrier is used when there are two or more rings. It represents a reference point in the cycle at which one phase in each ring must reach a common point of termination. In Figure D-2, a barrier is shown following phases 2 and 6. A second barrier is shown following phases 4 and 8. Between barriers, only one phase can be active at a time in each ring.

The ring structure dictates the sequence of phase presentation. Some common rules are provided in the following list:

- Phase pairs 1 and 2, 3 and 4, 5 and 6, and 7 and 8 typically occur in sequence. Thus, phase pair 1 and 2 begins with phase 1 and ends with phase 2. However, within each phase pair, it is possible to provide a lagging left turn phase by reversing the order of the pair. Thus, the pair 1 and 2 could be set to begin with phase 2 and end with phase 1 if it was desired to have the left-turn phase 1 lag through phase 2.
- Phase pair 1 and 2 can operate concurrently with phase pair 5 and 6. That is, phase 1 (or phase 2) can time with either phase 5 or 6. Phase pair 3 and 4 can operate concurrently with phase pair 7 and 8. These phase pairs are also known as concurrency groups.
- For a given concurrency group, the second phase to occur in one phase pair must end at the same time as the second phase to occur in the other pair (i.e., end at the barrier).
- Phases between the two barriers are typically assigned to the movements on a common road.
Operational Modes

There are three operational modes for the turn movements at an intersection. The names used to describe these modes refer to the manner in which the turn movement is served by the controller. The three modes are:

- Permissive
- Protected
- Protected-Permissive

The permissive mode requires turning drivers to yield to conflicting traffic streams before completing the turn. Permissive left-turning drivers yield to oncoming vehicles and permissive right-turning drivers yield to pedestrians. The efficiency of this mode is dependent on the availability of gaps in the conflicting streams. An exclusive turn lane may be provided, but it is not required. The permissive turn movement is typically presented with a circular green indication (although some agencies use other indications, such as a flashing yellow arrow). The right-turn movements in Figure D-2 are operating in the permissive mode.

The protected mode allows turning drivers to travel through the intersection while all conflicting movements are required to stop. This mode provides for efficient turn movement service; however, the additional turn phase typically results in increased delay to the other movements. An exclusive turn lane is typically provided with this mode. The turn phase is indicated by a green arrow signal indication. Left-turn movements 3 and 7 in Figure D-2 are operating in the protected mode.

The protected-permissive mode represents a combination of the permissive and protected modes. Turning drivers have the right-of-way during the exclusive phase. They can also complete the turn “permissively” when the adjacent through movement receives its circular green indication. This mode provides for efficient turn movement service, often without causing a significant increase in the delay to other movements. Left-turn movements 1 and 5 in Figure 2 are operating in the protected-permissive mode.

In general, the operational mode used for one left-turn movement is also used for the opposing left-turn movement. For example, if one left-turn movement is permissive, so is the other left-turn. However, this agreement is not required.

Left-Turn Phase Sequence

This section describes the sequence of service provided to left-turn movements, relative to the other intersection movements. The typical options include:

- No Left-Turn Phase (Permissive-Only)
- Leading Left-Turn
- Lagging Left-Turn
- Split
The permissive-only option is used when the left-turn operates in the permissive mode. A left-turn phase is not provided with this option. An illustrative implementation of permissive-only phasing for left- and right-turning traffic is shown in Figure D-3 for the minor street.

Figure D-3. Illustrative lag/lag and permissive-only phasing.

Leading, lagging, or split phasing is used when the left-turn operates in the protected or protected-permissive mode. The terms “leading” and “lagging” indicate the order with which the left-turn phase is presented, relative to the conflicting through movement. Leading left-turn phasing was shown previously in Figure D-2 for both left-turn movements on both the major and minor streets. Lagging left-turn phasing is shown in Figure D-3 for both left-turn movements on the major street. A mix of leading and lagging phasing (called lead/lag) is shown in Figure D-4 for the left-turn movements on the major street.

Figure D-4. Illustrative lead/lag and split phasing.
Split phasing describes a phase sequence where one phase serves all movements on one approach and a second phase serves all movements on the other approach. Split phasing requires that all approach movements simultaneously receive a green indication. Split phasing is shown in Figure D-4 for the minor street. Other variations exist and depend on the treatment of the pedestrian movements. The left-turn movement in a split phase typically operates in the protected mode (as shown), provided that there are no conflicting pedestrian movements.

HCM Actuated Signal Timing Estimation Procedure

The signalized intersection evaluation methodology in Chapter 16 of the HCM was developed with the expectation that phase interval times will be provided by the analyst. This expectation is reasonable when the signal system is operated in a pretimed mode. However, the green interval duration for an actuated phase is not something that is readily available to practitioners. Appendix B of Chapter 16 describes a procedure for estimating the average green interval duration.

Pretimed Control

For pretimed operation, a model for allocating time among the phases is described in Appendix B of HCM Chapter 16. This model is based on the distribution of cycle time in proportion to the critical phase flow ratios. It yields green interval durations that result in the same volume-to-capacity ratio for all critical movements. Other models for determining green interval durations are available in the literature. They typically attempt to optimize the timing such that some combination of delay, stops, and fuel consumption are minimized. These models are not addressed in this appendix.

Fully-Actuated Control

For fully-actuated control, a procedure is described in Appendix B of HCM Chapter 16 for estimating the average green interval duration. If the phase sequence includes more than one actuated phase, then the calculations are iterative. The duration of an actuated green interval is comprised of two time increments. The first increment represents the time required to clear the queue of vehicles. The second increment represents the time the green is extended by arriving vehicles. The second increment ends when there is a gap in traffic (i.e., gap out) or the green extends to the maximum limit (i.e., max out). Thus, the duration of the actuated phase is:

\[
D_p = l_1 + g_s + g_e + Y
\]

where,
- \(D_p\) = phase duration, s;
- \(l_1\) = start-up lost time, s;
- \(g_s\) = queue service time, s;
- \(g_e\) = green extension time, s; and
- \(Y\) = combined yellow change and red clearance interval duration (intergreen), s.
Coordinated-Actuated Control

Appendix B of HCM Chapter 16 describes how the procedure for fully-actuated control can be extended to coordinated-actuated control. However, the discussion is general and a description of the sequence of calculations, and associated equations, is not provided.

Green interval duration at coordinated-actuated intersections is influenced by a variety of factors. One of the more significant factors is whether the phase is coordinated or non-coordinated. The duration of the non-coordinated phases is dictated by traffic demand in much the same manner as it is for an actuated phase. The non-coordinated phase duration is also constrained by its force-off setting. A non-coordinated phase is also referred to as an “actuated” phase when it is clear that the context is with regard to coordinated-actuated operation.

The duration of the coordinated phases (typically, the two major-street through phases) is dictated by the cycle length and the force-off settings for the non-coordinated phases. These settings define the time points in the signal cycle by which each non-coordinated phase must end. The force-off settings are used to ensure that the coordinated phases receive a green indication for a specific time in the cycle (presumably, this time is synchronized with the coordinated phase time at the adjacent intersections such that traffic is progressed along the major street).

Two modes are commonly used to force off the non-coordinated phases, they are: “fixed” and “floating.” Floating force-off effectively retains any cycle time that is unused by the non-coordinated phases and returns it as an early return to the coordinated phase. In contrast, fixed force-off points allow unused cycle time to be used by the subsequent phase, regardless of whether it is a coordinated or non-coordinated phase. Most modern signal controllers automatically compute the force-off settings based on the phase splits and phase sequence entered by the engineer.

Maximum Allowable Headway

The detector layout in a traffic lane and its associated timing define a limiting time headway between successive vehicle activations (i.e., calls) that dictates when the controlling signal phase can end. This limiting headway is referred to as the maximum allowable headway (MAH). Call headways arriving to the controller at intervals shorter than the MAH will extend the green interval. The first call headway arriving at an interval longer than the MAH will result in gap out. A detector design does not truly have a unique MAH because of the random distribution of vehicle speeds in the traffic lane; however, a procedure is described herein for estimating a representative MAH for a given design based on the average speed of traffic.

The MAH is correlated with the safety and efficiency of the intersection. MAH values that are too long or too short can compromise safety, efficiency, or both. Exceptionally short MAH values tend to increase the frequency of premature gap out and cause frequent cycle failures. Exceptionally long MAH values may extend the green and unnecessarily delay vehicles served by other phases. In combination with advance detection designs, excessive green extension can lead to frequent max out and result in an increase in the frequency of rear-end crashes. In short, the optimum detection design would be one that yields a MAH that is short enough to ensure a “snappy”
operation but not so short that premature gap outs occur. MAHs that are typically found to be effective range from 3 to 6 s. Shorter values in this range apply to stop-line-only detection; larger values are more common when advance detection is used.

The MAH for a detector design serving a lane group represents the maximum allowable headway between successive calls from vehicles in that group. The MAH for the design is dependent on the number of detectors serving the lane group, the length of these detectors, and the distribution of vehicle speeds in the lane group.

The procedure for calculating the MAH for a detector design is based on three assumptions. The first assumption is that the design speed range for the advance detectors (if such detectors are used) will include the speeds of at least 70 percent of all vehicles in the lane group. The second assumption is that all advance detectors are designed to operate together such that a vehicle moving at a speed within the design speed range will maintain a continuous call for green as it traverses these advance detectors. The third assumption is that the time headway between successive calls is exponentially distributed, as described by the following distribution:

\[ p = 1 - \phi e^{-\lambda (MAH - \Delta)} \]  
\[ \lambda = \frac{\phi q}{1 - \Delta q} \]  
\[ \phi = e^{-b \Delta q} \]

where,  
\[ p = \text{probability of a call headway being less than the MAH;} \]  
\[ \phi = \text{proportion of free (unbunched) vehicles;} \]  
\[ \lambda = \text{flow rate parameter, veh/s;} \]  
\[ q = \text{average flow rate of traffic stream (including bunched and unbunched vehicles), veh/s;} \]  
\[ \Delta = \text{headway of bunched vehicle stream (= 1.5 s for single-lane lane group, 0.5 s otherwise), s;} \]  
\[ MAH = \text{maximum allowable headway that will maintain a call, s;} \]  
\[ b = \text{bunching factor (= 0.6, 0.5, 0.8 for lane groups with 1, 2, and 3 or more lanes, respectively).} \]

Equation 4 is obtained from Equation B 16-12 in the HCM (1). The suggested values for the bunching factor \( b \) and the headway of bunched vehicles \( \Delta \) listed with these equations were obtained from Exhibit B 16-6 of the HCM.

The next subsection describes a procedure for computing the MAH for each lane group served by a phase, as developed by Bonneson and McCoy (3). It addresses the situation where the only detection provided is that used near the stop line to ensure efficient queue service. If advance detection is used (possibly with stop line detection) to provide safe phase termination on high-speed approaches, then an alternative procedure for computing the MAH is provided in Reference (3). The second subsection describes a technique for combining lane group MAHs into an equivalent MAH for the phase. This technique is needed when a phase serves two or more lane groups.
MAH for Detection Designs with No Advance Detectors

The MAH for stop line detection can be calculated using the following equation:

\[ MAH = PT + CE + \frac{L_{ds} + L_v}{1.47 S_R} \]  \hspace{1cm} (5)

where,

- \( MAH \) = maximum allowable headway, s;
- \( PT \) = passage time setting (sometimes referred to as vehicle interval, extension interval, or unit extension setting), s;
- \( CE \) = call-extension setting for the stop line detection zone (if used), s;
- \( L_{ds} \) = length of the stop line detection zone, ft;
- \( L_v \) = detected length of vehicle, ft; and
- \( S_R \) = running speed on the intersection approach, mph.

The relationship between the variables in Equation 5 is shown in Figure D-5. The \( MAH \), \( PT \), and \( CE \) settings are multiplied by the average speed to convert all dimensions shown into distance.

\[ \text{Figure D-5. Maximum allowable headway for stop line detection.} \]

Equation 5 is derived for the typical case where the detector amplifier is operating in the presence mode. If it is operating in the pulse mode, then \( MAH \) would equal the passage time setting \( PT \). Also, the call-extension setting \( CE \) is not typically used, in which case its value is 0.0 s.

Equivalent Maximum Allowable Headway

For the case where there is more than one lane group served by a phase, the equivalent \( MAH \) must be calculated. Also, phases that end together at a barrier and that are specified in the controller as needing to gap out simultaneously (the default setting in most controllers) will need to have an equivalent \( MAH \) calculated.

The equivalent \( MAH \), when used in Equation 2, should yield the probability \( p \) that the combined calls to the phase from one or more groups will extend the green indication. Mathematically, this can be written as:

\[ p = 1 - (1 - p_1)(1 - p_2)\cdots(1 - p_m) \]  \hspace{1cm} (6)

where,
\[ p = \text{probability of a call headway being less than the } MAH; \text{ and} \]
\[ m = \text{number of lane groups served during the phase}. \]

Substitution of Equation 2 in Equation 6 yields:

\[ p = 1 - \left(\Phi_1 e^{-\lambda_1 (MAH_1 - \Delta)}\right) \left(\Phi_2 e^{-\lambda_2 (MAH_2 - \Delta)}\right) \cdots \left(\Phi_m e^{-\lambda_m (MAH_m - \Delta)}\right) \]

where,
\[ \lambda_i = \text{flow rate parameter for lane group } i \ (i = 1, 2, \ldots, m), \text{ veh/s; and} \]
\[ MAH_i = \text{maximum allowable headway for lane group } i, \text{ s.} \]

Equation 7 simplifies to:

\[ p = 1 - \Phi^* e^{-\lambda^* (MAH^* - \Delta^*)} \]

with
\[ \Phi^* = e^{-\sum \lambda_i \Delta_i} \]
\[ MAH^* = \frac{\sum \lambda_i MAH_i}{\lambda^*} \]

\[ \lambda^* = \sum \lambda_i \]

\[ \Delta^* = \frac{\sum \lambda_i \Delta_i}{\lambda^*} \]

where,
\[ \Phi^* = \text{combined proportion of free (unbunched) vehicles;} \]
\[ \lambda^* = \text{total flow rate parameter, veh/s;} \]
\[ MAH^* = \text{equivalent maximum allowable headway that will maintain a call, s; and} \]
\[ \Delta^* = \text{equivalent headway of bunched vehicle stream, s.} \]

The flow rate for each lane group should be based on the volume traversing its detected area. For example, the flow rate for a left-turn lane group would be based on the corresponding left-turn movement volume. Special consideration is needed when traffic in one lane group also activates some or all of the detectors in an adjacent lane group. This situation may occur when the length of an exclusive turn lane is less than the length of the detection zone for the adjacent through-movement lane group. In this situation, the turning flow will also contribute to the flow in the through lane group. When this situation occurs, it is recommended that the flow rate for the through-movement lane group be set equal to the flow rate entering its detection zone (i.e., the turning plus the through movement flow rate).
Green Extension Model

A noted previously with respect to Equation 1, the actuated phase duration is dictated by the time required to serve the queue and by the time the phase is extended by arriving traffic. The calculation of each of these two times is described in this section.

Queue Service Time

The queue service time for a through movement (or a protected left-turn movement) is computed as:

\[ g_s = f_q \frac{q_L C (1 - P)}{s - q_L C P / g} \]  

with,

\[ g = G - l_1 - l_2 + Y = G - l_1 + 2.0 \]  

where,

- \( g_s \) = queue service time, s;
- \( q_L \) = lane flow rate (= \( q / N \)), veh/s/ln;
- \( P \) = proportion of vehicles arriving on green (= \( R_p g / C \));
- \( C \) = cycle length, s;
- \( g \) = effective green duration, s;
- \( f_q \) = queue calibration factor (= 1.08 - 0.1 \( [G / G_{max}]^2 \));
- \( N \) = number of lanes in lane group serving the subject traffic stream;
- \( s \) = saturation flow rate of the lane group serving the subject traffic stream; veh/s/ln;
- \( G \) = green interval duration, s;
- \( G_{max} \) = maximum green setting for the subject phase, s;
- \( l_1 \) = start-up lost time, s;
- \( l_2 \) = end lost time, s (= \( Y - 2.0 \)); and
- \( Y \) = combined yellow change and red clearance interval duration (intergreen), s.

Equation 13 is equivalent to Equation B 16-8 in the HCM. The queue calibration factor accounts for “randomness in arrivals” (I). Hale and Courage (4) suggest that this adjustment is needed only when two or more movements are served concurrently.

Green Extension Time

As described by Bonneson and McCoy (3), the probability of a max out can be equated to the joint probability of there being a sequence of calls to the phase in service, each with a headway less than the \( MAH \). This probability can be stated mathematically as:

\[ p_x = p^n \]  

where,

- \( p_x \) = probability of phase termination by reaching the maximum green limit;
- \( p \) = probability of a call headway being less than the \( MAH \); and
- \( n_x \) = number of calls necessary to extend the green to max out.
In this application, the probability \( p \) is computed using Equation 8.

Equation 15 requires an estimate of the number of arrivals needed to extend the green to max out. This estimate can be obtained by dividing the maximum green duration by the average headway of all vehicles with headways less than the equivalent \( MAH \). The equations for estimating the number of arrivals and the average headway are:

\[
 n_x = \frac{G_{\text{max}} - MAH^* - (g_x + l_x)}{h} \geq 0.0 \tag{16}
\]

and,

\[
 h = \frac{\int_0^{MAH^*} t \varphi^* \lambda^* e^{-\lambda^*(t-\Delta^*)} \, dt + (1 - \varphi^*) \Delta^*}{\int_0^{MAH^*} \varphi^* \lambda^* e^{-\lambda^*(t-\Delta^*)} \, dt + (1 - \varphi^*)}
\]

\[
 = \frac{\Delta^* + \varphi^* \lambda^* - (MAH^* + 1/\lambda^*) \varphi^* e^{-\lambda^*(MAH^* - \Delta^*)}}{1 - \varphi^* e^{-\lambda^*(MAH^* - \Delta^*)}}
\]

where,

\( h = \) average headway for all vehicles with headways less than \( MAH \), s/veh.

**Green Extension Time**

The green extension time is based on the controller settings and the detection design. The minimum green setting for the subject phase ensures a green interval equal to the minimum value each time the phase is called. The maximum green setting limits the duration of green extension.

The average number of green extensions before the phase terminates is dependent on the probability \( p \) and the number of calls needed to extend the green to max out. The average number of extensions \( N \) can be calculated as:

\[
 N = \frac{\sum_{i=0}^{n-1} ip^i(1-p) + np^n}{\sum_{i=0}^{n-1} p^i(1-p) + p^n} \tag{18}
\]

with,

\[
 n = q_p^* \left[ G_{\text{max}} - (g_x + l_x) \right] \geq 0.0 \tag{19}
\]

\[
 q_p^* = \frac{1}{\Delta^* + 1/\lambda^*} \tag{20}
\]
where,
\[ N = \text{average number of green extensions}; \]
\[ n = \text{number of extensions before the green interval reaches its maximum limit}; \]
\[ q_p^* = \text{flow rate of the unbunched traffic stream, veh/s}. \]

The average number of extensions is used in the following equation to compute the average green extension time, given that at least one extending call arrives.

\[
g_e = \frac{N}{q_p^*} = \frac{p (1 - p^*)}{q_p^* (1 - p)}
\]

(21)

Substituting Equation 8 into Equation 21 and letting the term “\( p^* \)” equal 0.0 yields the following equation:

\[
g_e = \frac{e^{\lambda^* (MAH^* - \Delta)}}{\lambda^* q_p^*} - \frac{1}{q_p^*}
\]

(22)

Equation 22 is very similar to Equation B 16-9 in the HCM. However, the \( q_p^* \) variable in the denominator of the first term of Equation 22 is replaced by the phase flow rate \( q_p (= \Sigma q_i) \) in Equation B 16-9 and it is replaced by \( \lambda^* \) in the second term. The impact of this variation on the resulting estimate of \( g_e \) is small.

Equation B 16-9 does not include the “\( 1 - p^* \)” term in the numerator of Equation 21. This term accounts for the occasional green extension to the maximum limit. The impact of this omission can be significant if the phase frequently extends to its maximum green limit.

An examination of Equations 21 and 22 indicates that, as the phase flow rate \( q_p \) goes to zero, the average green extension converges to a value of 2.0 to 4.0 s (i.e., to the value \( MAH - \Delta[1 + b] \)). This tendency is illogical because it implies that the phase has some green extension even when there is no flow. Moreover, many low-volume actuated phases end almost immediately after the queue has cleared the stop line (with no extension). The green extension estimate obtained from Equation 21 actually represents the average extension when an extending call arrives. The extension occurs when the call arrives within a time equal to the \( MAH \) after queue service. The probability that a call arrives in this time interval is equal to \( p \). Thus, this probability is multiplied by Equation 21 to obtain the following equation for average green extension:

\[
g_e = \frac{p^2 (1 - p^*)}{q_p^* (1 - p)}
\]

(23)

The estimates obtained from Equation 13 and 23 are used in the following equation to compute the average green interval duration for a phase, given that it is called:
where, 
\[ G_{\text{call}} = \text{average green interval given that the phase is called, } s; \text{ and} \]
\[ G_{\text{min}} = \text{minimum green setting, } s. \]

The probability that the phase is called depends on whether it is set on recall in the controller. If it is on recall, then the probability that the phase is called equals 1.0. If the phase is not on recall, then the probability that it is called can be estimated using the following equation:

\[ p_c = 1 - e^{-q_p c} \]  \hspace{1cm} (25)

where,
\[ p_c = \text{probability that the subject phase is called;} \]
\[ q_p = \text{total flow rate in the subject phase } (= q_1 + q_2 + \ldots + q_m), \text{ veh/s;} \text{ and} \]
\[ m = \text{number of lane groups served during the phase.} \]

When single entry is specified for a phase (which is typically the default) in the controller, the flow rate used in Equation 25 represents the total flow rate in the subject phase \( q_p \). However, if dual entry is specified, then the flow rate of the two concurrent phases on the “other” ring must be added to \( q_p \).

The average green duration \( G \) is estimated using the following equation:

\[ G = G_{\text{call}} p_c \]  \hspace{1cm} (26)

If maximum recall is set for the phase then \( G \) is equal to \( G_{\text{max}} \). The average actuated phase duration \( D_p \) is obtained by adding \( G \) and \( Y \).

As indicated by Equations 13 and 14, the queue service time \( g_s \) is dependent on the green interval duration \( G \). This dependency introduces a circularity in the calculation process. Thus, the procedure is iterative, where an initial estimate of green interval duration is provided and the computations continued until the green interval duration is computed using Equation 26. The computed duration is compared with the initial estimate and, if they are different, then the computed value becomes the new initial estimate and the procedure is repeated. Iterations are repeated until the initial estimate and computed green interval duration are equal.

**DETERMINING SIGNAL PHASE DURATION**

This part of the appendix describes a procedure for determining phase duration for an intersection with coordinated-actuated signal control. It addresses the following conditions that may be present at an intersection with this type of control:

- Operational Mode
- Left-Turn Phase Sequence
- Force Mode
- Detection Design
The procedure is described in a narrative format and does not define every equation needed to compute average phase duration. This approach is taken because of the large number of equations required to address the full range of signalization and lane allocation conditions found at intersections in most cities. Nevertheless, all of these equations have been developed and are automated in the computational engine which is available from TRB’s Highway Capacity and Quality of Service Committee. Some of the equations presented in the previous part are repeated in this part for reader convenience.

Data Requirements

This section describes the data needed to use the procedure for determining signal phase duration. These data are listed in Table D-1. They are separated into the following six categories: traffic characteristics, geometry, left-turn phasing, phase assignments, controller settings, and detection. The data elements identified by asterisk “*” represent the additional data that are needed, relative to that required to use the methodology described in the main body of Chapter 16 of the HCM.

### TABLE D-1 Coordinated-actuated procedure input data requirements

<table>
<thead>
<tr>
<th>Data Category</th>
<th>Data Element</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic Characteristics</td>
<td>Volume</td>
<td>For each vehicular movement</td>
</tr>
<tr>
<td></td>
<td>Running speed*</td>
<td>Average speed on the intersection approach</td>
</tr>
<tr>
<td>Geometry</td>
<td>Number of lanes</td>
<td>For each vehicular movement</td>
</tr>
<tr>
<td>Left-Turn Phasing</td>
<td>Operational mode</td>
<td>Permissive, protected, protected-permissive</td>
</tr>
<tr>
<td></td>
<td>Phase sequence</td>
<td>Leading left-turn, lagging left turn, split</td>
</tr>
<tr>
<td></td>
<td>Sneakers*</td>
<td>Number of left turns at the end of a permissive phase</td>
</tr>
<tr>
<td>Controller Settings</td>
<td>Minimum green*</td>
<td>Shortest green time for a phase</td>
</tr>
<tr>
<td></td>
<td>Intergreen time</td>
<td>Yellow change plus red clearance intervals</td>
</tr>
<tr>
<td></td>
<td>Passage time*</td>
<td>Include any extend settings used for specific detector channels</td>
</tr>
<tr>
<td></td>
<td>Cycle length</td>
<td>Common cycle length for intersections in signal system</td>
</tr>
<tr>
<td></td>
<td>Phase recall*</td>
<td>Minimum recall, maximum recall</td>
</tr>
<tr>
<td></td>
<td>Entry mode*</td>
<td>Dual entry, single entry</td>
</tr>
<tr>
<td></td>
<td>Gap-out mode*</td>
<td>Simultaneous, independent</td>
</tr>
<tr>
<td></td>
<td>Phase splits*</td>
<td>Used to define force-offs for noncoordinated phases</td>
</tr>
<tr>
<td></td>
<td>Offset &amp; reference point*</td>
<td>Time coordinated phase begins (or ends) relative to master</td>
</tr>
<tr>
<td></td>
<td>Force mode*</td>
<td>Fixed, floating</td>
</tr>
<tr>
<td>Detection Design</td>
<td>Detection layout*</td>
<td>Distance from stop line</td>
</tr>
<tr>
<td></td>
<td>Detector length*</td>
<td>Length of detection zone</td>
</tr>
</tbody>
</table>

Note:
1 - Data elements indicated by asterisk “*” are considered additional data, relative to the data required by the methodology in Chapter 16 of the HCM (1).
As indicated in Table D-1, 12 additional data elements are needed to evaluate coordinated-actuated operation, relative to pretimed operation. These elements embrace the commonly used controller functions as well as those functions that have a significant effect on intersection operation. Additional data elements would be needed to evaluate actuated operation in greater detail or to incorporate additional controller functions. Functions not addressed by this procedure include:

- Gap reduction for actuated phases
- Variable initial for actuated phases
- Fully-actuated operation
- Pedestrian actuated operation
- A right-turn phase that overlaps with the complementary left-turn phase on the cross street
- Different split value for each phase in a given phase pair (i.e., they are assumed to have the same split)

In addition, the effect of the following factors on phase duration is not specifically addressed by the procedure:

- Short lanes or bays
- Demand starvation due to a closely-spaced upstream intersection
- Right turn on red
- Protected left-turn operation from a shared lane

The remainder of this section describes selected data elements listed in Table D-1.

**Traffic Characteristics**

**Running Speed.** The detector layout in a traffic lane and its associated timing define a limiting time headway between successive vehicle activations (i.e., calls) that dictates when the controlling signal phase can end. This limiting headway is referred to as the Maximum Allowable Headway (MAH). Call headways arriving to the controller at intervals shorter than the MAH will extend the green interval. The first call headway arriving at an interval longer than the MAH will result in gap out. A detector design does not truly have a unique MAH because of the random distribution of vehicle speeds in the traffic lane; however, a procedure is described herein for estimating a representative MAH for a given design based on the running speed of traffic. If this speed is not known, it can be estimated using the procedure described in Appendix B.

**Left-Turn Phasing**

**Operational Mode.** There are three operational modes for the left-turn movements at an intersection. The names used to describe these modes refer to the manner in which the left-turn movement is served by the controller. The three modes are:

- Permissive
- Protected
- Protected-Permissive
The permissive mode requires left-turning drivers to yield to the oncoming traffic stream before completing the turn. An exclusive left-turn lane may be provided, but it is not required. The permissive left-turn movement is typically presented with a circular green indication (although some agencies use other indications, such as a flashing yellow arrow).

The protected mode allows left-turning drivers to travel through the intersection while all conflicting movements are required to stop. An exclusive left-turn lane is typically provided with this mode. The turn phase is indicated by a green arrow signal indication.

The protected-permissive mode represents a combination of the permissive and protected modes. Left-turning drivers have the right-of-way during the protected left-turn phase. They can also complete the turn “permissively” when the adjacent through movement receives its circular green indication.

**Left-Turn Phase Sequence.** Typical left-turn phase sequences include:

- Permissive Only (i.e., no left-turn phase)
- Leading Left-Turn (with or without Dallas Display)
- Lagging Left-Turn (with or without Dallas Display)
- Split

The permissive-only option is used when the left-turn operates in the permissive mode. A left-turn phase is not provided with this option.

Leading, lagging, or split phasing is used when the left-turn operates in the protected or protected-permissive mode. The terms “leading” and “lagging” indicate the order with which the left-turn phase is presented, relative to the conflicting through movement. If both left-turn phases on a street lead their respective conflicting through movement, then the phasing is called “lead-lead.” If both left-turn phases lag their conflicting through movement, then the phasing is called “lag-lag.” If one left-turn phase leads and the other lags, then the phasing is called “lead-lag.”

When the protected-permissive mode is used with lead-lag or lag-lag phasing, then the “yellow trap” (or left-turn trap) problem may occur for one or both of the left-turn movements. This problem stems from the potential conflict between left-turn vehicles and oncoming vehicles at the end of the adjacent through phase. Of the two through movement phases serving the subject street, the trap is associated with the first through movement phase to terminate and occurs during this phase’s intergreen period. The left-turn driver seeking a gap in oncoming traffic during the through phase, first sees the yellow ball indication; then incorrectly assumes that the oncoming traffic also sees a yellow indication; and then turns across the oncoming traffic stream without regard to the availability of a safe gap. The “yellow trap” problem can be alleviated by using one of the following techniques:

- Use the protected mode for both left-turn movements.
- Use a “Dallas Display” for both left-turn signal heads and use the protected-permissive mode for both left-turn movements.
The second technique avoids the yellow trap by using an overlap in the controller and a five-section left-turn signal head. An overlap is a controller output (to the signal head load switch) that is associated with two or more phases. The left-turn green, yellow, and red ball indications are associated with the opposing and adjacent through movement phases using an overlap. The left-turn signal head uses louvers on the yellow and green ball indications to prevent through movement drivers from viewing the left-turn display. The louvered signal head is referred to as the “Dallas Display.” With this display, both left-turn phases can operate in the protected-permissive mode and the trap is avoided.

Split phasing describes a phase sequence where one phase serves all movements on one approach and a second phase serves all movements on the other approach of the same street. Split phasing requires that all approach movements simultaneously receive a green indication. The left-turn movement in a split phase typically operates in the protected mode, provided that there are no conflicting pedestrian movements. The phase duration procedure accommodates split phasing on the street that does not serve the coordinated movements.

**Phase Assignments.** The assignment of movements to phases is an input to the signal controller. A typical movement numbering scheme is indicated in Figure D-1. The typical assignment of these movements to the basic eight controller phases is illustrated in Figures D-2, D-3, and D-4. Many other assignments are possible, depending on the conditions present at the intersection, the desired phase sequence, and the preferences of the operating agency. However, to simplify the mechanics of the procedure, phase assignments are limited to those shown in Table D-2.

**TABLE D-2  Phase assignments supported by the phase duration procedure**

<table>
<thead>
<tr>
<th>Movement Type</th>
<th>Movement Number</th>
<th>Assigned Phase Number based on Left-Turn Phase Sequence for the Approach</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Permissive Mode</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permissive Only</td>
</tr>
<tr>
<td>Left</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>Through + Right</td>
<td>2 and 2R</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>4 and 4R</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>6 and 6R</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>8 and 8R</td>
<td>8</td>
</tr>
</tbody>
</table>

Note:
1 - For lagging left-turn phase sequences, the phase pair sequence is reversed.

Phase assignments do not have to be directly specified by the analyst. The actual assignment follows from Table D-2. It is based on the operational mode and left-turn phase sequence that is specified by the analyst. For example, consider the westbound approach in Figure D-1. It serves
left-turn movement 1 and through movement 6. If left-turn movement 1 is operated in a permissive
mode (i.e., no left-turn phase), then Table D-2 indicates that it will be assigned to phase 6. Through
and right-turn movements 6 and 6R, respectively, are also assigned to this phase. If split phasing is
used on one approach, it must also be used on the opposing approach.

**Sneakers.** This variable describes the number of left turns that can be completed at the end
of a phase, when that phase operates in the permissive mode. The supplemental permissive left-turn
worksheet provided in Appendix C of Chapter 16 of the *HCM* includes an equation for estimating
sneakers. Values obtained from this equation range from 1 to 2, with the latter value typical of left-
turns from an exclusive lane. For permissive-only left turn operation, the actuated controller
operation is such that the phase is extended until waiting left-turn vehicles have cleared. Thus,
sneakers only occur when the permissive-only phase has terminated by max out. For protected-
permissive operation, the left-turn detection is not monitored during the permissive operation. Thus,
sneakers can occur following the permissive portion of the protected-permissive operation when a
left-turn queue is present.

**Controller Settings**

**Phase Recall.** Recall is a phase-specific input. It applies only to the noncoordinated phases.
If used for a specific phase, it places a recurring call for service to this phase when it is not showing
a green indication. This ensures that the phase will display a green indication each cycle. The recall
can be set as “minimum vehicle recall” or “maximum vehicle recall.” Minimum vehicle recall
ensures that the phase displays a green for its minimum duration. Maximum vehicle recall ensures
that the phase displays a green for its maximum duration. Pedestrian recall is not supported but can
be replicated by selecting minimum vehicle recall and setting the minimum green equal to the
pedestrian walk plus clearance time.

**Entry Mode.** This mode is a phase-specific setting. It can be specified as “dual entry” or
“single entry.” Entry mode dictates controller operation when there is: (1) a call for service in a
phase associated with one ring but not in any of the concurrent phases in the other ring, and (2) a
barrier must be crossed to serve this call. With dual entry, one of the concurrent phases in the ring
that does not have a call is selected for activation, provided that one or both of the concurrent phases
have dual entry set. If both concurrent phases are single entry, then neither phase is activated.

**Gap-Out Mode.** This mode is a phase-specific setting; however, it is typically set the same
for all phases that serve the same street. It is set to simultaneous gap out as a default in most modern
controllers. This mode dictates controller operation when a barrier must be crossed to serve the next
call and one phase is active in each ring. It requires that both phases reach a point of being
committed to terminate (via gap out, max out, or force off) at the same time. If simultaneous gap
out is disabled, one phase can reach this point first (but remain green) before the other phase. In this
situation, the first phase to commit to termination does not change its status while waiting for the
other phase to commit to termination. Regardless of which mode is in effect, the barrier is not
crossed until both phases are committed to terminate.
Phase Splits. Each non-coordinated phase is provided a “split” time. A phase split represents the sum of the green, yellow change, and red clearance intervals for the phase. The rationale for determining the appropriate green interval duration varies among agencies; however, it is often equated to the optimum fixed green duration that is obtained from a signal timing optimization software program. The phase split for each phase in a concurrent pair (i.e., 1 and 5, 3 and 7, 4 and 8) is not required to be equal in most modern controllers; however, it is typically set at the same value. It is assumed in this procedure that each phase in the pair has the same split value (split values may vary among pairs).

Offset and Reference Point. The offset entered in a controller represents the time that the first coordinated phase begins (or ends) relative to the system master time zero. It must be specified as being referenced to the beginning, or the end, of the first coordinated phase. The offset reference point is typically the same at all intersections in a given signal system. The first coordinated phase is the coordinated phase that occurs first within the concurrent group of phases containing the coordinated phases when there are constant calls on all phases. The yield point for each coordinated phase occurs at the end of its green interval.

Force Mode. This mode is a controller-specific setting. It is set to “fixed” or “floating.” The controller calculates the phase force-off point for each actuated phase based on the force mode and the phase splits. When set to the fixed mode, each actuated phase has its force-off point set at a fixed time in the cycle, relative to time zero on the system master. This operation allows unused split time to revert to the following phase. When set to the floating mode, each actuated phase has its force-off point set at the split time after the phase first becomes active. This operation allows unused split time to revert to the coordinated phase via an early return to green.

Coordination Mode. This mode is a controller-specific setting. It describes the manner in which the coordinated phases are terminated to service calls on the actuated phases. In the simplest of modes, any call for service by an actuated phase that is received prior to the yield point terminates the coordinated phase at the yield point. In more complicated variations of this mode, a sliding window approach is used to sequentially allow service only to the “next” actuated phase with the benefit that additional green is provided to the end of the coordinated phase if there are no calls for service. These modes are not standardized in the controller industry. The procedure described in this section is based on the use of a permissive period that follows the yield point. It operates in a manner that maximizes the time available to accept calls for service on the actuated phases.

Detection Design

Detection Layout. The detection layout for an approach with both advance detection and stop line detection is shown in Figure D-6. If advance detection is not provided, then only the length of the stop line detection zone $L_{ds}$ is needed. If advance detection is used, then the location of each advance detector $D_{ai}$, as well as the location of the stop line detection zone $SB$, is needed.

Detector Length. The length of all detectors that serve the subject lane group is an input to the procedure. If inductive loop detection is used, the advance detectors are typically 6 ft in length and the stop line detection zone is typically 40 ft in length. The length of the stop line detection zone
is usually paired with the controller passage time such that they provide efficient queue service. A 40 ft stop line detection zone is typically paired with a passage time of 1.5 to 2.0 s. Longer stop line detection zones are sometimes used, in which case, the passage time would be shorter than 1.5 s.

![Diagram of detection layout with advance and stop line detection]

Figure D-6. Example detection layout with advance and stop line detection.

If video image vehicle detection is used, then the equivalent length of roadway that is monitored by the video detector is needed as input to the procedure. This length can be affected by the capability of the video detector product and it may vary among agencies.

**Volume Computations**

The procedure for estimating the average phase duration consists of completing the analysis steps outlined in this section and the next four sections. This section focuses on the volume computations needed to estimate the call rate to extend, and to call, a phase.

**Determine Call Rate to Extend Green**

The phase call rate represents the flow rate of calls to the controller channel assigned to a specific phase. This rate dictates the green extension time for the phase.

A. **Determine Lane Group Volume** ($v_{LG}$). A phase can serve one or more lane groups. The volume of each lane group associated with a phase is dependent on the type of movement, approach lane allocation, and the presence of advance detection. The following rules apply when determining the lane groups served by a phase and their volume:

1. If the phase exclusively serves a left-turn movement, then the phase’s lane group volume is equal to the left-turn movement volume.
2. If the phase serves a through or right-turn movement and there is no exclusive left-turn phase for the adjacent left-turn movement, then:

\[
D_i = \text{distance to the leading edge of advance detector } i, \text{ as measured from the stop line, ft};
\]

\[
L_{ds} = \text{length of the stop line detection zone};
\]

\[
n = \text{number of advance detectors};
\]

\[
SB = \text{distance between the trailing edge of the stop line detection zone and the nearest edge of the crossing travel path};
\]

\[
SL = \text{distance between the stop line and the nearest edge of the crossing travel path}.
\]
a. If there is an adjacent left-turn bay, then the phase’s left-turn lane group volume equals
the left-turn movement volume and the phase’s through lane group volume is equal to
the through and right-turn movement volume.

b. If there is no adjacent left-turn lane, then the phase’s through lane group volume equals
the approach volume. No other lane groups exist.

3. If the phase serves a through movement and there is an exclusive left-turn phase for the
adjacent left-turn movement, then the phase’s through lane group volume equals the sum of
the through and right-turn movement volumes. No other lane groups exist.

* Note: a left-turn bay is required for this procedure when the left-turn movement has an
exclusive phase.

4. If split phasing is used, then:
   a. If there is an adjacent left-turn lane, then the phase’s left-turn lane group volume equals
      the left-turn movement volume and the phase’s through lane group volume is equal to
      the through and right-turn movement volume.
   b. If there is no adjacent left-turn lane, then the phase’s through lane group volume equals
      the approach volume. No other lane groups exist.

Once the lane group volume associated with each phase is determined using the above rules,
it may need to be modified if simultaneous gap out is enabled. If lagging left-turn phasing is used,
then the left-turn phase ends at the barrier; otherwise, the through phase ends at the barrier. Thus,
if lag/lag phasing is used on the major street, then phases 1 and 5 end at the barrier. If lead/lag
phasing is used on the major street, then phases 1 and 6 (or 2 and 5, depending on which approach
has lagging left-turn phasing) end at the barrier. If lead/lead left-turn phasing or no left-turn phasing
is used on the major street, then phases 2 and 6 end at the barrier.

The following rules should be evaluated to determine the modified lane group volumes if
simultaneous gap out is enabled. They are described for the case where phases 2, 6, 4, and 8 end at
the barrier. The rules should be modified if other phase pairs end at the barrier:

1. If phases 2 and 6 have simultaneous gap out enabled then the lane groups associated with
   phase 2 are also assigned to phase 6. Similarly, the lane groups associated with phase 6 are
   also assigned to phase 2.
2. If phases 4 and 8 have simultaneous gap out enabled then the lane groups associated with
   phase 4 are also assigned to phase 8. Similarly, the lane groups associated with phase 8 are
   also assigned to phase 4.

When simultaneous gap out is enabled, each phase that terminates at a barrier could each
have as many as two lane groups assigned to it. If two phases terminate at the barrier, then as many
as four lane groups could dictate the green extension time. The four groups would include the left-
turn and through lane groups served by the phase and the left-turn and through groups served by the
concurrent phase. The collective set of calls from these four lane groups effectively dictate the green
extension time for the phase.
B. Determine Lane Group Lanes ($n_{lg}$). The number of lanes associated with each lane group identified in Step A is determined in this step. This determination is made based on the geometry of the subject approach. By definition, a lane group has 1 or more lanes.

C. Determine Extending Call Rate ($\lambda$). The objective of this step is to compute the extending call rate for each phase. This rate is effectively equal to the total flow rate of the unbunched vehicle stream $\lambda^*$. It is computed as:

$$\lambda^* = \sum_{i=1}^{m} \lambda_i$$  \hspace{1cm} (27)

with,

$$\lambda_i = \frac{\phi_i q_i}{1 - \Delta_i q_i}$$  \hspace{1cm} (28)

where,

$\lambda^*$ = total call rate parameter, veh/s;
$\lambda_i$ = call rate parameter in lane group $i$, veh/s;
$\phi_i$ = proportion of free (unbunched) vehicles in lane group $i$;
$q_i$ = average flow rate of traffic stream in lane group $i$ (= $v_i/3600$), veh/s;
$v_i$ = volume for lane group $i$, veh/h;
$\Delta_i$ = headway of bunched vehicle stream (= 1.5 s for single-lane lane group, 0.5 s otherwise), s;
m = number of lane groups served during the phase; and
$b_i$ = bunching factor (= 0.6, 0.5, 0.8 for lane groups with 1, 2, and 3 or more lanes, respectively).

At the start of this step, the appropriate value of $\Delta$ and $b$ is established for each lane group identified in Step A. The value for each factor is based on the number of lanes in the lane group, as indicated in the variable list. Then, the proportion of free vehicles is computed for each lane group using Equation 29. Next, the flow rate parameter for each lane group is computed using Equation 28. Finally, the parameters are added using Equation 27 to obtain the call rate parameter associated with the phase.

It is also useful to compute the following two variables for each phase during this step. These variables will be used in a subsequent step to compute the green extension time.

$$\phi^* = e^{-\sum_{i=1}^{m} b_i \Delta_i q_i}$$  \hspace{1cm} (30)

$$\Delta^* = \frac{\sum_{i=1}^{m} \lambda_i \Delta_i}{\lambda^*}$$  \hspace{1cm} (31)

where,

$\phi^*$ = combined proportion of free (unbunched) vehicles; and
$\Delta^*$ = equivalent headway of bunched vehicle stream, s.
Determine Call Rate to Activate a Phase

The phase activation call rate is used to determine the probability of a phase being activated such that it provides green to one or more waiting vehicles. It is based on the arrival rate of calls to the phase while it is not green. These calls come from the stop line detection when this detection is used. If it is not used, then it is assumed that locking memory is used with advance detection.

A. Determine Phase Volume \((v_P)\). The volume associated with a phase is dependent on the type of movements it serves and the approach lane allocation. The following rules apply when determining the phase volume:

1. If the phase exclusively serves a left-turn movement, then the phase volume is equal to the left-turn movement volume.
2. If the phase serves a through or right-turn movement and there is no exclusive left-turn phase for the adjacent left-turn movement, then the phase volume equals the approach volume.
3. If the phase serves a through or right-turn movement and there is an exclusive left-turn phase for the adjacent left-turn movement, then the phase volume equals the sum of the through and right-turn movement volumes.
   • Note: a left-turn bay is required for this procedure when the left-turn movement has an exclusive phase.
4. If split phasing is used, then the phase volume is equal to the approach volume.

B. Determine Activation Call Rate \((q_P)\). The activation call rate is equal to the phase volume after dividing it by 3600 to convert it to units of vehicles per second. If dual entry is activated for a phase then its activation call rate must be modified by adding its original rate to that of both concurrent phases. For example, if phase 2 is set for dual entry then the modified phase 2 activation call rate equals the original phase 2 activation call rate plus the activation rate of phase 5 and the activation rate of phase 6. In this manner, phase 2 is activated when demand is present for phases 2, 5, or 6.

Queue Accumulation Polygon

This section describes the calculations needed to construct the queue accumulation polygon (QAP) associated with each lane group served during a phase. This diagram defines the average queue length for a traffic movement as a function of time during the average cycle. The shape of the polygon is defined by the arrival flow rate during the effective red and green intervals, saturation flow rate associated with each movement in the lane group, signal indication status, left-turn operation mode, and phase sequence. Once constructed, the polygon can be used to compute the queue service time and control delay for the corresponding lane group.

The QAP for a through movement is shown in Figure D-7. It shows the polygon for the average cycle. The red and green intervals are ordered from left to right in the sequence of presentation, such that the last two time periods correspond to the queue service time \(g_s\) and green extension time \(g_e\) of the subject phase. The variables shown in the figure are defined in the following list:
\( \text{g} = \) effective green time, s;
\( \text{r} = \) effective red time, s;
\( \text{g_s} = \) queue service time (\( = Q_r / [s - q_g] \)), s;
\( \text{g_e} = \) green extension time, s;
\( \text{q} = \) average flow rate of traffic stream, veh/s;
\( \text{q_L} = \) lane group flow rate per lane (\( = q / \text{N} \)), veh/s/ln;
\( \text{q_r} = \) flow rate during the effective red time (\( = [1 - \text{P}] q_L C / r \)), veh/s/ln;
\( \text{q_g} = \) flow rate during the effective green time (\( = \text{P} q_L C / \text{g} \)), veh/s/ln;
\( \text{Q_r} = \) queue size at the end of effective red time (\( = q_r \text{r} \)), veh/s/ln;
\( \text{N} = \) number of lanes;
\( \text{P} = \) proportion of vehicles arriving on green (\( = \text{R}_p \text{g} / \text{C} \));
\( \text{C} = \) cycle length, s; and
\( \text{s} = \) saturation flow rate; veh/s/ln.

If the lane group flow rate \( q_L \) exceeds the lane capacity (\( = C / [s \text{ g}] \)), then it is set equal to the lane capacity.

![Queue accumulation polygon for protected movements.](image)

At the start of the effective red, vehicles begin to queue at a rate of \( q_r \) and accumulate to a length of \( Q_r \) vehicles at the time the effective green begins. Thereafter, the queue begins to discharge at a rate of \( s - q_g \) until it clears \( g_e \) seconds before the end of the phase. The queue service time \( g_s \) represents the time required to serve the queue present at the end of effective red \( Q_r \) plus any additional arrivals that join the queue before it fully clears. As suggested by Figure D-7, this queue service time can be computed as \( Q_r / [s - q_g] \). Substitution of the variable relationships in the variable list into this equation yields Equation 13, shown previously (with \( f_q = 1.0 \)).

A triangular shape is formed by the queue arrival and discharge processes. The height of the triangle at any instant in time represents the queue length. The total delay incurred is represented by the area under the triangle. For this polygon, and subsequent polygons shown, the procedure described in of Chapter 16 of the HCM is used to estimate the saturation flow rate \( s \).
The polygon in Figure D-7 applies to through lane groups or left-turn lane groups served during a left-turn phase that operates in the protected-only mode. This polygon is also applicable to split phasing. For split phasing, each approach is evaluated separately to determine the queue service time and green extension time for the corresponding phase. If the approach has a left-turn lane, then a separate polygon is constructed for both the left-turn and the through lane group. The larger queue service time of the two lane groups dictates the duration of the queue service time for the phase.

More complicated combinations of phase sequence and left-turn operational mode dictate more complicated queue accumulation polygons. A polygon must be uniquely derived for each combination. The most common combinations are illustrated in Exhibits B16-8, 9, 10, and 11 in Appendix B of Chapter 16 of the HCM. These exhibits are reproduced in Figures D-8, D-9, D-10, and D-11. The variables shown in these figures are defined in the previous list, with the following additions:

- $g_p$ = effective green time for permissive left-turn operation, s;
- $g_u$ = duration of $g_p$ that is not blocked by an opposing queue, s;
- $g_f$ = time before the first left-turn vehicle arrives and blocks the shared lane, s;
- $g_l$ = effective green time for left-turn phase, s;
- $s_p$ = saturation flow rate of permissive left-turn operation, veh/s/ln;
- $p_x$ = probability of phase termination by reaching the maximum green limit;
- $P_L$ = proportion of left-turn vehicles in the inside lane of a shared lane approach;
- $Q_q$ = queue size at the start of $g_u$, veh;
- $Q_p$ = queue size at the end of permissive service time, veh;
- $Q_p'$ = queue size at the end of permissive service time, adjusted for sneakers, veh; and
- $Q_f$ = queue size at the end of $g_f$, veh.

Figure D-8. Polygon for permissive-only left turns from exclusive lane.
Figure D-9. Polygon for permissive-only left turns from shared lane.

Figure D-10. Polygon for protected-permissive operation with leading left-turn phase.

Figure D-11. Polygon for protected-permissive operation with lagging left-turn phase.
The polygon in Figure D-8 applies to the left-turn lane group that operates as permissive only during the adjacent through phase. Figure D-7 applies to the adjacent through lane group. The lane group with the longer queue service time dictates the queue service time for the phase. If the phase extends to max out such that $g_e$ is equal to 0.0, then some left-turns will be served as sneakers.

The polygon in Figure D-9 applies to shared lane approaches. If the approach has two or more lanes, the polygon in Figure D-9 applies to the inside lane serving left-turn and through vehicles. Figure D-7 is used for the outside lanes. If the phase extends to max out such that $g_e$ is equal to 0.0, then some left-turns will be served as sneakers. The expected number of sneakers is computed as $(1 + P_L)$, where $P_L$ is the proportion of left-turn vehicles in the inside lane.

The polygon in Figure D-10 applies to left-turn movements that have protected-permissive operation with a leading left-turn phase. The polygon in Figure D-11 applies to the same movements and operation but with a lagging left-turn phase. If a queue exists at the end of the permissive period for either polygon, then the queue is reduced by the number of sneakers.

The remaining discussion in this section describes the sequence of calculations needed to construct the queue accumulation polygon. This sequence may vary, depending on the left-turn operational mode, phase sequence, and lane assignments for the subject lane group.

A. Determine Permissive Green Time ($g_u$). This step is applicable when left-turns are served using the permissive or protected-permissive mode. The permissive green time for a left-turn movement represents the time during the opposing through phase that occurs after its queue service and before yellow onset. This time is available to the left-turn movement for permissive operation. The permissive green time can be influenced by phase sequence, as described by the following rules:

1. If permissive-only operation exists for the subject left-turn, then its permissive green time is equal to the opposing through phase duration minus the start-up lost time, queue service time, and intergreen time for the opposing through phase.
2. If protected-permissive operation exists and “Dallas Display” is not used (the typical case), then:
   a. If lead/lead phasing is provided, then the permissive green time for the left-turn movement with the shorter left-turn phase duration is equal to the smaller of: (1) the sum of the left-turn and through phase durations less the opposing left-turn phase duration and subject phase intergreen time, and (2) the unqueued green time for the opposing through movement. The permissive green time for the left-turn movement with the longer left-turn phase duration is equal to the unqueued green time for the opposing through movement.
   b. If lead/lag phasing is provided, then the permissive green time for the left-turn movement that is served in the lagging left-turn phase is equal to the opposing through green interval less the larger of the leading left-turn phase duration or the queue service time of the opposing through movement. The permissive green time for the left-turn movement that is served in the leading left-turn phase is equal to the adjacent through green interval less the leading left-turn phase duration and the queue service time of the opposing through phase.
c. If lag/lag phasing is provided, then the permissive green time for each left-turn movement is equal to the smaller of the two through green intervals less the queue service time for the opposing through phase.

3. If protected-permissive operation exists and “Dallas Display” is used, then the permissive green time is equal to the unqueued green time for the opposing through movement.

B. Determine Time Before First Left-Turn Arrives \((g_1)\). This step is applicable when left turns are served using the permissive mode on a shared lane approach. This variable represents the time that elapses from the start of permissive green to the arrival the first left-turn vehicle at the stop line. During this time, through vehicles in the inside shared lane depart at a rate of \(s\). Guidance for computing this variable is provided in Appendix C of Chapter 16 of the HCM.

C. Determine Permissive Left-Turn Saturation Flow Rate \((s_h)\). This step is applicable when left-turns are served using the permissive or protected-permissive modes. This saturation flow rate is computed using the following equation:

\[
s_h = v_o \frac{e^{-v_o t_c / 3600}}{1 - e^{-v_o t_f / 3600}}
\]  

(32)

where,
\(s_h\) = saturation flow of permissive left turns, veh/h/ln;
\(v_o\) = opposing through-and-right-turn phase volume, veh/h;
\(f_{LUo}\) = lane utilization factor for opposing flow;
\(t_c\) = critical gap (= 4.5 s), s; and
\(t_f\) = follow-up headway (= 4.5 s if the subject left-turn is served in a shared lane, 2.5 s if the subject left-turn is served in an exclusive lane), s.

This equation, and the recommended constants, is specified in Exhibit C 16-3 in Chapter 16 of the HCM.

D. Determine Through Car Equivalent \((E_{LL})\). This step is applicable when left-turns are served using the permissive or protected-permissive modes. For shared lanes, this variable is computed as:

\[
E_{LL} = \frac{s_o}{s_h} - 1
\]  

(33)

where,
\(s_o\) = base saturation flow rate (= 1900), pc/h/ln.

For exclusive left-turn lanes, this variable is computed as:

\[
E_{LL} = \frac{s_o}{s_h}
\]  

(34)

E. Determine Proportion of Left-Turns in Lane \((P_L)\). This step is applicable when left-turns are served using the permissive mode. It describes the proportion of left-turn vehicles in the
shared lane. Guidance for computing this variable is provided in Appendix C of Chapter 16 of the
HCM.

F. Determine Saturation Flow Rate of Permissive Operation (s_p). This step is applicable
when left-turns are served using the permissive or protected-permissive modes. For left-turns served
by an exclusive lane, saturation flow rate of permissive left-turn operation s_p is equal to s_{lt}/3600.
For left-turns served by a shared lane, the procedure in Appendix C of Chapter 16 is used to estimate
the saturation flow rate for the inside lane as s_p = s_{fm}/3600, where f_m is the left-turn adjustment
factor for the inside lane of a shared lane approach. Guidance in this appendix (see Equation C 16-4)
indicates that the saturation flow rate for each outside lane is estimated as 0.91 s.

G. Define Queue Accumulation Polygon. During this step, the times and saturation flow
rates computed in the previous steps are used to construct the queue accumulation polygon. The
arrival rate during red and during green are computed using the equations in the variable list
associated with Figure D-7. The appropriate polygon shape is selected from Figures D-7 through
D-11 based on the left-turn mode of operation, phase sequence, and lane assignments. The queue
length Q is computed for each point on the polygon where the arrival rate or discharge rate changes.

Maximum Allowable Headway

The procedure for calculating the MAH for the detection associated with a phase is described
in this section. Step A computes the MAH when stop line detection is provided (but no advance
detection). Step B describes a technique for combining lane group MAHs into an equivalent MAH
for the phase. This technique is needed when a phase serves two or more lane groups or when
simultaneous gap out is enabled.

A. Determine MAH. The MAH for a lane group served by stop line detection can be
calculated using the following equation:

\[
MAH = PT + \frac{L_{ds} + L_v}{1.47 S_R}
\]

where,

\(MAH\) = maximum allowable headway for the stop line detection zone, s;

\(PT\) = passage time setting (sometimes referred to as vehicle interval, extension interval, or unit
extension setting), s;

\(L_{ds}\) = length of the stop line detection zone, ft;

\(L_v\) = detected length of vehicle, ft; and

\(S_R\) = running speed on the intersection approach, mph.

Equation 35 is derived for the typical case where the detector amplifier is operating in the
presence mode. If it is operating in the pulse mode, then MAH would equal the passage time setting
PT.

B. Determine Equivalent Maximum Allowable Headway (MAH*). The equivalent MAH
is calculated for cases where there is more than one lane group served by a phase. It is also
calculated for phases that end at a barrier and that are specified in the controller as needing to gap out simultaneously with a phase in the other ring (the default setting in most controllers). The rules described in this step are written in a generic manner so that they can be applied to all phases. These rules are outlined in the following paragraphs:

1. If simultaneous gap out is not enabled or the phase does not end at the barrier, then:
   a. If the phase serves only one movement, then the $MAH^*$ for the phase equals the $MAH$ computed for the corresponding lane group.
   b. If the phase serves the left-turn and through (or right-turn) movements and there is not an exclusive left-turn phase for the approach (i.e., it operates as permissive only), then:
      i. If there is no left-turn lane group (i.e., a shared lane), then the $MAH^*$ for the phase is computed as:

\[
MAH^* = \frac{v_{th} MAH + v_{lt} (MAH + 3600/s_{lt} - t_f^*)}{v_{th} + v_{lt}} 
\]

where,
- $v_{th} = \text{volume for through and right-turn movements, veh/h}$;
- $v_{lt} = \text{volume for left-turn movement, veh/h}$; and
- $MAH = \text{maximum allowable headway for the lane group, s}$.

The term “$3600/s_{lt} - t_f^*$” in Equation 36 represents the additional time needed to serve the permissive left-turning vehicles. These vehicles move slowly over the detectors and effectively increase the $MAH$ duration, relative to that computed using the average approach speed.

   ii. If there is a left-turn lane group, then the $MAH^*$ for the phase is computed as:

\[
MAH^* = \frac{\lambda_{th} MAH_{th} + \lambda_{lt} (MAH_{lt} + 3600/s_{lt} - t_f^*)}{\lambda_{th} + \lambda_{lt}} 
\]

where,
- $MAH_{th} = \text{maximum allowable headway for the through lane group, s}$;
- $MAH_{lt} = \text{maximum allowable headway for the left-turn lane group, s}$;
- $\lambda_{th} = \text{call rate parameter (= extending call rate) for the through lane group, veh/s}$; and
- $\lambda_{lt} = \text{call rate parameter (= extending call rate) for the left-turn lane group, veh/s}$.

   c. If the phase serves all approach movements using split phasing, then:
      i. If there is one lane group (i.e., a shared lane), then the $MAH^*$ for the phase equals the $MAH$ computed for the lane group.
      ii. If there is a left-turn and through lane group, then the $MAH^*$ for the phase is computed as:

\[
MAH^* = \frac{\lambda_{th} MAH_{th} + \lambda_{lt} MAH_{lt}}{\lambda_{th} + \lambda_{lt}} 
\]

\[\text{(38)}\]
2. If simultaneous gap out is enabled and the phase ends at the barrier, then the preceding rules are applied to each intersection approach and the $MAH^*$ for the phase is computed as:

$$MAH^* = \frac{\lambda_{s,i} MAH_{s,i} + \lambda_{c,j} MAH_{c,j} + \lambda_{s,i} MAH_{s,i} + \lambda_{c,j} MAH_{c,j}}{\lambda_{s,i} + \lambda_{c,j} + \lambda_{s,i} + \lambda_{c,j}}$$

(39)

where,

$MAH_{s,i} =$ maximum allowable headway for subject lane group $i$, s;

$MAH_{c,j} =$ maximum allowable headway for the concurrent lane group $j$, s;

$\lambda_{s,i} =$ call rate parameter (= extending call rate) for the subject lane group $i$, veh/s; and

$\lambda_{c,j} =$ call rate parameter (= extending call rate) for the concurrent lane group $j$, veh/s.

When there is split phasing, there are no concurrent phases and Equation 39 defaults to Equation 38.

Equivalent Maximum Green

In coordinated-actuated operation, the force-off points are used to constrain the duration of the actuated (non-coordinated) phases. The maximum green timer is also available to provide additional constraint; however, it is not commonly used. In fact, the default mode in most modern controllers is to inhibit the maximum green timer when the controller is used in a coordinated signal system.

The relationship between the force-off points, yield point, and phase splits is shown in Figure D-12. The yield point is associated with the coordinated phases (phases 2 and 6 are typically used as the coordinated phases). It coincides with the start of the yellow change interval and the beginning of the permissive period. If a call for service by one of the actuated phases arrives after the yield point is reached, then the coordinated phases begin the termination process by presenting the yellow indication. The permissive period duration is dependent on the coordination mode used. It typically is sufficiently long as to ensure that a call for an actuated phase is served, if there is time before the force-off point is reached to present a green indication for its minimum duration.

One force-off point is associated with each of the concurrent phase pairs 3 and 7, 4 and 8, and 1 and 5. If either phase of a pair is extended to its force-off point, then the phase begins the termination process. Modern controllers compute the force-off points and yield point using the entered phase splits and intergreen periods based on the relationships shown in Figure D-12.

The procedure described in this appendix computes an equivalent maximum green for each actuated phase to replicate the effect of the force-off on phase duration. The method of estimating this maximum green varies by force mode. Both methods are described in the next two steps.

A. Determine Equivalent Maximum Green for Floating Force Mode ($G_{\text{max}}$). This step is only applicable if the controller is set to operate in the floating force mode. With this mode, each actuated phase has its force-off point set at the split time after the phase first becomes active. The force-off point for a phase is established when the phase is first activated. Thus, the force-off point “floats,” or changes, each time the phase is activated. This operation allows unused split time to revert to the coordinated phase via an early return to green. The equivalent maximum green for this
mode is computed as being equal to the phase split less the intergreen period. This relationship is shown in Figure D-12 for phases 4 and 8.

Figure D-12. Relationship between force-off points, yield point, and phase splits.

B. Determine Equivalent Maximum Green for Fixed Force Mode ($G_{\text{max}}$). This step is only applicable if the controller is set to operate in the fixed force mode. With this mode, each actuated phase has its force-off point set at a fixed time in the cycle, relative to time zero on the system master. The force-off points are established whenever a new timing plan is selected (e.g., by time of day) and remains “fixed” until a new plan is selected. This operation allows unused split time to revert to the following phase.

The equivalent maximum green for this mode is computed for each phase. It is computed by first establishing the fixed force-off points (as shown in Figure D-12) and then computing the average duration of each actuated phase. The maximum green for a specific phase is computed as the difference between its force-off point and the sum of the previous actuated phases. Equation 40 illustrates this computation for phase 4. A similar calculation is performed for the other actuated phases.
\[ G_{\text{max,4}} = FO_4 - (YP_2 + Y_2 + G_3 + Y_3) \]  

where,
- \( G_{\text{max,4}} \) = equivalent maximum green for phase 4, s;
- \( FO_4 \) = force-off point for phase 4, s;
- \( YP_2 \) = yield point for phase 2, s;
- \( G_i \) = green interval duration for phase \( i \), s; and
- \( Y_i \) = intergreen period (yellow change interval plus red clearance interval) for phase \( i \), s.

The maximum green obtained from this equation is shown in Figure D-13 for the ring that serves phases 1, 2, 3, and 4. Unlike Figure D-12, Figure D-13 illustrates the actual average phase durations for a given cycle. In this example, phase 3 timed to its minimum green and terminated. It never reached its force-off point. The unused time from phase 3 is made available to phase 4 and results in a larger maximum green than obtained using the floating mode (see Figure D-12). If every actuated phase extends to its force-off point, then the maximum green from the fixed force mode equals that obtained from the floating force mode.

![Figure D-13. Example equivalent maximum green for fixed force mode.](image_url)
Average Phase Duration

This section describes the sequence of calculations needed to estimate the average duration of a phase. In fact, the process requires the calculation of the duration of all phases because of the constraints imposed by the controller ring structure and associated barriers.

Advance knowledge of the green interval duration is required in the calculation of queue service time, permissive green time, left-turn volume served during the permissive portion of a protected-permissive left-turn operational mode, and equivalent maximum green. This advance knowledge introduces a circular dependency that requires an iterative process to solve. The green interval for each phase is initially estimated and then the procedure is implemented using this estimate. When completed, the procedure provides a new initial estimate of the green interval duration. The calculations are repeated until the initial estimate and computed green interval duration are equal.

The calculation steps that comprise the procedure are described in the following paragraphs:

A. Estimate Green Interval \((G)\). An initial estimate of the green interval duration is provided for each phase. The estimate should at least equal the minimum green setting associated with the phase. Typically, the initial estimate is set equal to the input phase splits less the intergreen period.

B. Compute Equivalent Maximum Green \((G_{\text{max}})\). The equivalent maximum green is computed for each actuated phase, based on the estimated green interval duration, phase splits, and intergreen times. The procedure for computing \(G_{\text{max}}\) is described in the section titled, “Equivalent Maximum Green.”

C. Construct the Queue Accumulation Polygon. The queue accumulation polygon is constructed for each lane group and corresponding phase. If the left-turn movement is served by a shared lane on a multilane approach, then separate polygons are constructed for the inside lane and for the remaining lanes. The procedure for computing the arrival rates, discharge rates, and times is described in the section titled, “Queue Accumulation Polygon.”

D. Compute Queue Service Time \((g_s)\). The queue service time is computed for each queue accumulation polygon constructed in the previous step. For through movements, or left-turn movements served during a left-turn phase operating in the protected-only mode, the polygon in Figure D-7 applies and the following equation can be used:

\[
g_s = \frac{q_L C (1 - P)}{s - q_L C P |g|} \tag{41}
\]

where,

\(g_s\) = queue service time, s;
\(q_L\) = lane flow rate, veh/s/ln;
\(P\) = proportion of vehicles arriving on green (= \(R_p\) g/C);
\(C\) = cycle length, s;
The saturation flow rate $s$ used in Equation 41 is obtained from Equation 16-4 of Chapter 16 of the HCM.

E. **Compute the Extending Call Rate** ($\lambda$). The extending call rate is computed for each actuated phase, based on the flow rate associated with each lane group and the number of lanes served in each lane group. The procedure for computing $\lambda$ is described in the section titled, “Determine Call Rate to Extend Green.”

F. **Compute Equivalent Maximum Allowable Headway** ($MAH^*$). The equivalent maximum allowable headway is computed for each actuated phase, based on the lane group $MAH$s served during the phase and the corresponding extending call rate for each lane group $\lambda$. The procedure for computing the $MAH^*$ is described in the section titled, “Maximum Allowable Headway.”

G. **Compute Number of Arrivals Before Max Out** ($n$). The average number of arrivals before the green extends to max out is computed for each actuated phase using the following equation:

$$n = q_p^* \left[ G_{\text{max}} - (g_s + l_p) \right] \geq 0.0$$  \hspace{1cm} (42)

with,

$$q_p^* = \frac{1}{\Delta^* + 1/\lambda^*}$$  \hspace{1cm} (43)

H. **Compute Probability of Green Extension** ($p$). The probability of the green interval being extended by randomly arriving vehicles is computed for each actuated phase using the following equation:

$$p = 1 - q_p^* e^{-\lambda^* (\Delta H^* - \Delta^*)}$$  \hspace{1cm} (44)

I. **Compute Green Extension Time** ($g_e$). The average green extension time is computed for each actuated phase using the following equation:

$$g_e = \frac{p^2 (1 - p)}{q_p^* (1 - p)}$$  \hspace{1cm} (45)

J. **Compute the Activation Call Rate** ($q_p$). The call rate to activate a phase is computed for each actuated phase, based on the phase volume and the use of dual entry. The procedure for computing $q_p$ is described in the section titled, “Determine Call Rate to Activate a Phase.”

K. **Compute Probability of Phase Call** ($p_c$). The probability that an actuated phase is called depends on whether it is set on recall in the controller. If it is on recall, then the probability
that the phase is called equals 1.0. If the phase is not on recall, then the probability that it is called can be estimated using the following equation:

\[ p_c = 1 - e^{-q_r c} \]  \hspace{1cm} (46)

L. **Compute Unbalanced Green Duration (\(G_u\)).** The unbalanced green duration is computed for each actuated phase using the following equation:

\[ G_u = \text{G}_{\text{call}} p_c \leq G_{\text{max}} \]  \hspace{1cm} (47)

with,

\[ \text{G}_{\text{call}} = \text{larger of:} \left[ \frac{l_1 + g_s + g_t}{G_{\text{min}}} \right] \]  \hspace{1cm} (48)

where,

\( G_{\text{min}} = \) minimum green setting, s.

If maximum recall is set for the phase then \( G_u \) is equal to \( G_{\text{max}} \).

The green duration obtained from this step is "unbalanced" because it does not reflect the constraints imposed by the controller ring structure and associated barriers. These constraints are imposed in a subsequent step.

M. **Compute Unbalanced Phase Duration (\(D_{\text{up}}\)).** The unbalanced average phase duration \( D_p \) is computed for each actuated phase by adding the unbalanced green duration \( G_u \) and the corresponding intergreen time \( Y \).

N. **Compute the Average Phase Duration (\(D_p\)).** It is assumed that phases 2 and 6 are the coordinated phases serving movements 2 and 6, respectively (see Figure D-1) for this discussion. If the left-turns operate in the protected-permissive or protected-only modes, then the opposing left-turn movements are served during phases 1 and 5. If a coordinated phase occurs first in the phase pair, then the other phase (i.e., the one serving the opposing left-turn movement) is a "lagging" left-turn phase (see Figure D-4 for example).

The following rules are used to estimate the average duration of each phase:

1. **If the phase is associated with the street serving the coordinated movements, then:**
   a. If a left-turn phase exists for the subject approach, then its duration \( D_{p,lt} \) equals \( D_{\text{up},lt} \) and the opposing through phase has a duration \( D_{\text{up},th} \) based on the following equation:

\[ D_{p,t} = C - \text{Larger of:} \left( D_{\text{up}3} + D_{\text{up}4} + D_{\text{up}5} + D_{\text{up}6} \right) - D_{p,lt} \]  \hspace{1cm} (49)

where,

\( D_{\text{up},i} = \) unbalanced phase duration for phase \( i \), s.

b. If a left-turn phase does not exist for the subject approach, then \( D_{p,lt} \) equals 0.0 and the equation above is used to estimate the duration of the one phase that serves all movements on the opposing approach.
The procedure for determining average phase duration only accommodates split phasing on the street that does not serve the coordinated movements.

If \( D_{P,i} \) obtained from the equation above is less than the minimum phase duration (= \( G_{\text{min}} + Y \)), then the phase splits are too generous and do not leave adequate time for the coordinated phases.

2. If the phase is associated with the street serving the noncoordinated movements, then:
   a. If an approach is served by two concurrent phases (i.e., phase \( a \) followed by phase \( b \)), then the duration of \( D_{P,a} \) is equal to unbalanced phase duration of the first phase to occur \( D_{uP,a} \). The duration of \( D_{uP,b} \) is based on the following equation:

   \[
   D_{P,b} = \text{Larger of} \ (D_{uP,b} + D_{uP,b} + D_{uP,b} - D_{P,a})
   \]

   For example, if the concurrent phase pair consists of phase 3 followed by phase 4 (i.e., a leading left-turn arrangement), then \( D_{P,3} = D_{uP,3} \) and \( D_{P,4} \) is computed from the equation above. In contrast, if the pair consists of phase 8 followed by phase 7 (i.e., a lagging left-turn arrangement), then \( D_{P,8} = D_{uP,8} \) and \( D_{P,7} \) is computed from the equation above.

   b. If an approach is served with one phase operating in the permissive-only mode (but not split phasing), then \( D_{P,a} = 0.0 \) and the equation above is used to estimate the duration of the one phase that serves all approach movements.

   c. If split phasing is used, then \( D_{P,a} \) equals the unbalanced phase duration for one approach and \( D_{P,b} \) equals the unbalanced phase duration for the other approach.

O. Compute the Green Interval Duration (\( G \)). The average green interval duration is computed for each phase by subtracting the intergreen time \( Y \) from the average phase duration \( D_P \).

P. Compare Computed and Estimated Green Interval Durations. The green interval duration computed in the previous step is compared with the value estimated in Step A. If the two values are not equal, then the computed green interval becomes the “new” initial estimate and the sequence of calculations is repeated starting with Step B. This iterative process is repeated until the two green intervals are equal, in which case the average green interval duration represents the last computed green interval duration.

SENSITIVITY ANALYSIS

This part of the appendix examines the sensitivity of average green interval duration to various traffic volume conditions. Specifically, the examination considers the effect of traffic volume and phasing on the computed green interval duration. The procedure described in the previous part of the appendix was used to estimate the average green interval duration for two intersections on a coordinated street segment.

The evaluation testbed was based on the S.W. Barbur Boulevard street segment located between S.W. 19th Avenue and S.W. Bertha Boulevard. This segment is 2937 ft in length, has two through lanes in each direction, and a speed limit of 35 mph. The left-turn movements for the coordinated street at both intersections were operated using the protected mode and led the through phases. The left-turn movements on S.W. 19th Avenue were operated using the protected-permissive mode and led the through phases. Split phasing was used on S.W. Bertha Boulevard. The cycle
length was 100 s. Progression was established for the peak travel direction and was characterized as Arrival Type 4. Progression for the off-peak direction was characterized as Arrival Type 3.

Sixteen volume scenarios were evaluated. The procedure was used to estimate the average green interval duration for each phase at each intersection. A total of 224 averages were obtained in this manner (= 8 phases x 16 volume combinations at one intersection + 6 phases x 16 volume combinations at the other intersection).

Figure D-14 shows the estimated average green interval durations for the actuated phases. The averages are related to the volume associated with each phase in the figure. Each data point represents one volume combination for one phase at one intersection. The data shown using open circles represents the first actuated phase to occur in each phase pair (i.e., the left-turn phases) and the split phases serving S.W. Bertha Boulevard. The ring structure of the actuated controller requires the duration of these phases to be dictated by the corresponding phase volume (within the constraints of minimum green, maximum green, and force off). Thus, the correlation between volume and green duration is expected. Almost the entire green interval for these phases consists of queue service time. Only a small portion of the green is due to vehicle extension due to the relatively low volume of the associated movements.

Figure D-14. Effect of phase volume on green duration of actuated phases.

The solid triangle and square data points correspond to the second actuated phase to occur in the phase pair that serves S.W. 19th Avenue (i.e., phases 4 and 8). The ring structure of the actuated controller requires the duration of the “critical” phase pair (i.e., the pair whose combined duration is longer than that of the concurrent pair) to each be dictated by the corresponding phase volume. This relationship is described in the text associated with Equation 50. The critical phase pair at this intersection consists of phases 7 and 8. The green duration for the phase that occurs
second in this pair (i.e., phase 8) is shown in Figure D-14 using the solid triangle data points. Its strong correlation with volume reflects its designation as a “critical” phase.

The square data points in Figure D-14 correspond to the second actuated phase serving S.W. 19th Avenue that is non-critical (i.e., phase 4 of the phase pair 3 and 4). As suggested by Equation 50, the duration of the non-critical phase is dependent on the duration of the “other” three phases (i.e., phases 3, 7, and 8). Hence, the duration of the non-critical phase is not dependent on its associated volume. This lack of correlation with volume is shown in Figure D-14 for the square data points.

Figure D-15 shows the estimated average green interval durations for the two coordinated phases (i.e., phases 2 and 6) at each intersection. The solid data points represent the green intervals for the coordinated phases at S.W. 19th Avenue (i.e., Intersection 2). The open circle data points represent the green intervals at S.W. Bertha Boulevard (i.e., Intersection 1). There is little correlation with phase volume because the coordinated phase duration is dictated by the phase splits and any cycle time that is unused by the actuated phases. The phase splits for Intersection 1 yield 46 s of green for the coordinated phase. The phase splits for Intersection 2 yield 50 s of green for the coordinated phase. The data points for each intersection equal or exceed these values. The amount by which they exceed the green split represents unused cycle time by the actuated phases.

![Figure D-15. Effect of phase volume on green duration of coordinated phases.](image)

**MODEL CALIBRATION**

This part of the appendix summarizes the findings from the calibration of the phase duration procedure. The calibration was based on a comparison of the computed green interval duration with the average green interval obtained from the CORSIM traffic simulation model (version 5.1).
The simulation testbed was based on the S.W. Barbur Boulevard street segment located between S.W. 19th Avenue and S.W. Bertha Boulevard. This segment was described in the Sensitivity Analysis part.

Sixteen volume scenarios were each simulated for one hour for the S.W. Barbur segment. The green interval duration for each phase at each intersection was recorded each cycle using post-processing software developed for this project. The post-processor scanned the animation file produced by CORSIM and extracted the desired data. The animation file contains a second-by-second record of signal status for each simulation run. The green interval duration for each phase and cycle were then used to compute the average green interval duration for each simulation run. A total of 224 averages were obtained in this manner (= 8 phases x 16 hours at one intersection + 6 phases x 16 hours at the other intersection).

Figure D-16 compares the computed green interval durations with the averages obtained from the simulation runs. Each data point represents one hour of simulation for one phase at one intersection. The thick black trend line represents the best-fit regression line. The equation for this line, and its coefficient of determination $R^2$, are shown in the figure. The slope coefficient is not significantly different from 1.0 and the intercept coefficient is not significantly different from 0.0 s. These findings lead to the conclusion that the procedure can be used to estimate the average green interval duration without bias and that it does not need to include additional calibration constants.

![Figure D-16. Comparison of computed and simulated green interval durations.](image)

The root mean square error for the regression line is 1.2 s. This variable provides an indication of the standard deviation of the prediction. It is based on one hour of simulation for each data point. Smaller values are likely for longer simulation runs.
REFERENCES


APPENDIX E

Procedure for Estimating Stop Rate at a Signalized Intersection
APPENDIX E

PROCEDURE FOR ESTIMATING STOP RATE AT A SIGNALIZED INTERSECTION

INTRODUCTION

This appendix describes a procedure for estimating the full stop rate at a signalized intersection. The procedure is sufficiently general that it can be used to estimate the total number of stops, which includes those vehicles that complete either a full or a partial stop. The procedure can be used to estimate the stop rate for a specified intersection lane group. The lane group stop rate can be aggregated for all lane groups to obtain an overall approach or intersection stop rate.

The procedure requires as input the duration of the phase serving the lane group as well as the signal cycle length. If the control type is coordinated actuated, then the procedure described in Appendix D can be used to estimate the average phase duration.

This appendix consists of four parts. The first part provides a review of relevant models for estimating stop rate. The second part describes the procedure for determining stop rate. The third part describes the findings from a sensitivity analysis using the procedure. The last part summarizes the findings from the model calibration activity.

BACKGROUND

This part of the appendix reviews two stop rate prediction models described in the literature. Each model is described in a separate section. Both models were developed for application to a through movement or a protected-only left-turn movement in an exclusive lane. The concepts of full versus partial stop are discussed. Also discussed is the implication of having different flow rates during the effective red and effective green times associated with a phase. The third section extends the models described in the literature to the more general case of full stop prediction for actuated traffic movements with various left-turn phase sequences and operational modes (e.g., protective-permissive).

Stop Rate Model for Uniform Arrivals

The average number of full stops per vehicle at an intersection is defined as the full stop rate (1). A full stop is defined to occur when a vehicle slows to zero (or a crawl speed, if in queue) in response to a change in signal indication from green to red. The distinction between a full and a partial stop is shown in Figure E-1. This figure illustrates the trajectory of each vehicle as it traverses an intersection approach during one signal cycle. There is no overflow at the end of the cycle shown in this figure.

Each thin line in Figure E-1 that angles upward from left to right represents the trajectory of one vehicle. The time between trajectories represents the headway between vehicles (i.e., the inverse
of its flow rate \( q \). The slope of the trajectory represents the vehicle’s speed. The curved portion of a trajectory indicates deceleration or acceleration. The horizontal portion of a trajectory indicates a stopped condition. The effective red \( r \) and effective green \( g \) times are dimensioned at the top of the figure. The other variables shown are defined in the subsequent discussion.

**Figure E-1. Time-space diagram of vehicle trajectory on an intersection approach.**

Figure E-1 shows the trajectories of eight vehicles. The first five trajectories (counting from left to right) have a horizontal component to their trajectory that indicates they have reached a full stop as a result of the red indication. The sixth trajectory has some deceleration and acceleration but the vehicle does not stop. This trajectory indicates a partial stop was incurred for the associated vehicle. The last two trajectories do not incur deceleration or acceleration, and the associated vehicles do not slow or stop. Thus, the number of full stops \( N_f \) is 5 and the number of partial stops \( N_p \) is 1. The total number of stops \( N_t \) is 6. The full stop rate is 0.63 stops/veh (= 5/8).

Akcelik (1) indicates that the full stop rate can be estimated using the following equation:

\[
h = \left( 0.9 \frac{1 - \frac{g}{C}}{1 - \frac{q_L}{s}} \right) + \left( 0.9 \frac{N_o}{qC} \right) \tag{1}
\]

where,

- \( h \) = full stop rate, stops/veh;
- \( q_L \) = lane group flow rate per lane (= \( q/N \)), veh/s/ln;
- \( q \) = lane group flow rate, veh/s;
- \( C \) = cycle length, s;
- \( g \) = effective green duration, s;
- \( N \) = number of lanes in lane group;
- \( s \) = saturation flow rate of the lane group; veh/s/ln; and
- \( N_o \) = overflow queue for the lane group, veh.
Equation 1 assumes that the arrival flow rate is random (i.e., uninfluenced by the presence of an upstream signalized intersection) such that it can be modeled as uniform arrival headway distribution. The equation used to compute the overflow queue $N_o$ is similar to that used in the HCM (2) to estimate the average overflow queue $Q_i$ (i.e., Equation G 16-9 in Appendix G of Chapter 16).

The first term in Equation 1 represents the proportion of vehicles stopped once by the signal. The second term represents the additional stops that may occur during overflow (i.e., cycle failure) conditions. The contribution of this term becomes significant when the volume-to-capacity ratio exceeds about 0.8. The constant of “0.9” is an empirical adjustment that is used to exclude partial stops. The full stop rate typically varies from 0.4 stops/veh at low volume-to-capacity ratios to 2.0 stops/veh when the volume-to-capacity ratio is about 1.0.

**Stop Rate Model for Platooned Arrivals**

Olszewski (3) extended Equation 1 to the situation where the arrival flow rate can vary during the signal cycle, such as when platoons are formed by an upstream signalized intersection. His extension was based on characterizing arrival flows into two categories: those vehicles that arrive during the green indication and those vehicles that arrive when the signal indication is not green. Thus, a flow rate during the green $q_g$ is computed and a flow rate during the remainder of the cycle is computed $q_r$. This characterization is used in Chapter 16 of the HCM to estimate control delay for signalized intersections in coordinated signal systems.

The model developed by Olszewski is applicable to traffic movements served in exclusive lanes and in a protected manner (i.e., a through movement or a protected-only left-turn movement). The equations that comprise the model are:

$$h = h_1 + \frac{N_o}{qC}$$

with,

$$h_1 = \frac{1 - P(1 + \frac{d_a}{g})}{1 - PX} : \text{If } d_a \leq (1 - P)gX$$

$$h_1 = \frac{(1 - P)(r - \frac{d_a}{g})}{r - (1 - P)gX} : \text{If } d_a > (1 - P)gX$$

$$d_a = 0.5 (1.47 S_R) (1/r_a + 1/r_d)$$

where,

- $h_1 =$ deterministic stop rate, stops/veh;
- $P =$ proportion of vehicles arriving on green ($= R_p g/C$);
- $r =$ effective red duration ($= C - g$), s;
- $X =$ volume-to-capacity ratio ($= q_L C/[s g]$);
- $d_a =$ deceleration-acceleration delay, s;
- $r_a =$ acceleration rate, ft/s²;
\[ r_d = \text{deceleration rate, ft/s}^2; \text{ and} \]
\[ S_r = \text{running speed, mph}. \]

The interpretation of the two terms in Equation 2 is analogous to that offered for the two terms in Equation 1. The deceleration-acceleration delay \( d_a \) term is shown in Figure E-1. It is used in Equations 3 and 4 to distinguish between a fully and a partially stopped vehicle. This delay term represents the time required to decelerate to a stop and then accelerate back to the initial speed, less the time that it would have taken to traverse the equivalent distance at the initial speed. Deceleration-acceleration delay values measured by Olszewski (3) range from 8 to 10 s (3).

**Stop Rate Model Extensions**

This section describes several extensions to the model developed by Olszewski (3). One extension provides an alternative to Equation 5 for estimating the acceleration-deceleration delay for the more general case where a “stopped” vehicle is defined as one that is moving at, or less than, a specified slow speed while in queue. The second extension describes how Equation G16-9 in Appendix G of Chapter 16 of the HCM can be used to estimate the average overflow queue for a lane group. A third extension describes how Equation G 16-9 can be extended to model overflow for an actuated intersection when the average green interval is obtained from the procedure described in Appendix D.

**Acceleration-Deceleration Delay**

Various definitions are used to describe when a vehicle is “stopped” for the purpose of field measurement. These definitions typically allow the observed vehicle to be called “stopped” even if it has a small nominal speed (say, 2 to 5 mph) while moving up in queue. Many stochastic simulation programs also have a similar allowance. These practical considerations in the count of stopped vehicles require the specification of a threshold speed that can be used to identify when a vehicle is effectively stopped. The following equation can be used to estimate the acceleration-deceleration delay associated with a specified threshold speed.

\[
d_a = \frac{[1.47 (S_r - S_s)]^2}{2 (1.47 S_r^2)} (1/r_d + 1/r_a) \tag{6}
\]

where,
\[ S_s = \text{threshold speed defining a stopped vehicle, mph}. \]

If the threshold speed \( S_s \) is set to 0.0, then Equation 6 reduces to Equation 5 and Equation 2 yields the full stop rate for vehicles whose speed reaches 0.0 mph. If the threshold speed is set to equal the running speed \( S_r \), then \( d_a \) equals 0.0 s and Equation 2 yields an estimate of the total stop rate (including both full and partial stops).
Overflow Queue for Lane Group

Equation G 16-9 in the HCM provides an estimate of the average overflow queue for the typical lane in a lane group. This equation is reproduced as Equation 7 below:

\[
N_o = 0.25 \ c_L T \left[ \left( X - 1 + \frac{Q_{bl}}{c_L T} \right) + \left( X - 1 + \frac{Q_{bl}}{c_L T} \right)^2 + \frac{8 k_b X}{c_L T} + \frac{16 k_b Q_{bl}}{(c_L T)^2} \right]
\]

with,

\[k_b = 0.10 \ I (s g)^{0.5}\]

where,

- \(N_o\) = average overflow queue per lane, veh/ln;
- \(c_L\) = lane group capacity per lane (= 3600 s g/C), veh/h/ln;
- \(T\) = length of analysis period, h;
- \(X\) = volume-to-capacity ratio (= \(q_L\) C/[s g]);
- \(I\) = upstream filtering factor for platooned arrivals (from Exhibit 15-7 of the HCM);
- \(k_b\) = second-term incremental factor; and
- \(Q_{bl}\) = initial queue per lane at the start of the analysis period, veh/ln.

The adjustment factor in Equation 8 is used for actuated signal phases. An alternative equation is provided in the HCM for pretimed signal phases. This alternative equation is appropriate to use when estimating the overflow queue for a coordinated phase.

Equation 7 estimates the number of vehicles in the typical lane of the lane group. To obtain the total number of queued vehicles in all lanes of the lane group, the capacity used in Equation 7 should reflect the capacity of the lane group. Thus, each instance of the variable \(c_L\) in Equation 7 should be multiplied by the number of lanes in the lane group \(N\) to obtain the desired estimate.

Available Capacity

Equation 7 is derived from a queue theory model that assumes a constant capacity (i.e., service time). For this reason, it is appropriate for modeling overflow queues at a pretimed intersection with fixed phase duration. However, simulation data indicate that Equation 7 tends to overestimate the overflow queue for an actuated phase when the average phase duration is used to estimate lane group capacity. Unlike a pretimed phase, an actuated phase has an “adaptive” capacity because it maintains the green indication until the queue is served on a cycle-by-cycle basis, regardless of how the demand may vary from cycle to cycle. Only when the actuated phase reaches its maximum green limit can an overflow possibly occur. Thus, an actuated phase minimizes the occurrence of an overflow queue (i.e., cycle failure). Equation 8 does not account for this behavior.

To account for the ability of an actuated phase to prevent overflow, the capacity used in Equation 7 should be based on the larger of the average green interval and the maximum green limit. This calculation is shown in the following equation:

\[
c_a = 3600 \ \frac{g_a s N}{C}
\]

with,
where,
\[ c_a = \text{lane group capacity available to an actuated phase, veh/h}; \]
\[ g_a = \text{available effective green time, s}; \]
\[ G_{\text{max}} = \text{maximum green setting for the subject phase, s}; \]
\[ l_1 = \text{start-up lost time, s}; \]
\[ l_2 = \text{end lost time, s (=} Y - 2.0); \]
\[ Y = \text{combined yellow change and red clearance interval duration (intergreen), s}. \]

The capacity \( c_a \) obtained from Equation 9 should be used in replacement of \( c_L \) in Equation 7 to estimate the overflow queue for the purpose of estimating the stop rate for an actuated phase. This modification is appropriate when the average green interval duration used is that obtained using the procedure described in Appendix D. The inclusion of the number of lanes variable \( N \) in Equation 9 accommodates the extension identified in the discussion associated with Equation 7.

**Incremental Delay**

The lane group capacity available for an actuated phase, as defined in Equation 9, should also be used in the incremental delay equation in Chapter 16 of the HCM (i.e., Equation 16-12) when the average green interval duration is obtained using the method described in Appendix D.

**Arrival-Departure Polygon**

The arrival-departure polygon (ADP) associated with a phase is a useful tool for deriving an equation for computing the number of full stops \( N_f \) in a traffic lane. This diagram defines the cumulative number of arrivals and departures associated with a traffic movement as a function of time during the average cycle. It is related to the queue accumulation diagram (QAP) but not identical. The main difference is that the polygon sides in the ADP represent an arrival or a discharge rate, but not both. In contrast, the polygon sides in the QAP represent the combined arrival and discharge rates that may occur during a common time interval. The ADP is useful for estimating the stop rate, while the QAP is useful for estimating delay. Either polygon can be used to estimate the queue service time.

The ADP for a through movement is shown in Figure E-2. The figure shows the profile for the average cycle. The red and green intervals are ordered from left to right in the sequence of presentation, such that the last two time periods correspond to the queue service time \( g_s \) and green extension time \( g_e \) of the subject phase. The variables shown in the figure were defined previously, with the following additions:

\[ t_r = \text{service time for fully stopped vehicles, s}; \]
\[ N_f = \text{number of fully stopped vehicles, veh}; \]
\[ g_s = \text{queue service time (=} Q_r / [s - q_g]), s}; \]
\[ g_e = \text{green extension time, s}; \]
\[ q_r = \text{flow rate during the effective red time (=} [1 - P] q_L C / r), \text{veh/s/ln}}; \]
\( q_s = \) flow rate during the effective green time \( (= P \frac{q_L}{C/g}) \), veh/s/ln; and 
\( Q_r = \) queue size at the end of effective red time \( (= q_r r) \), veh/ln.

**Figure E-2. Arrival-departure polygon.**

The higher solid trend line in Figure E-2 corresponds to vehicles arriving to the intersection. The lower solid trend line corresponds to queued vehicles departing the stop line. It is horizontal during the effective red, denoting no departures. The vertical distance between these two lines at any instant in time represents the queue length.

At the start of the effective red, vehicles begin to queue at a rate of \( q_r \) and accumulate to a length of \( Q_r \) vehicles at the time the effective green begins. Thereafter, the rate of arrival is \( q_g \) until the end of the effective green period. The queue service time \( q_s \) represents the time required to serve the queue present at the end of effective red \( Q_r \) plus any additional arrivals that join the queue before it fully clears. The dashed line in this figure represents only those vehicles that complete a full stop. The dashed line corresponding to arrivals to the stopped queue lags behind the solid arrival line by \( d_a / 2 \) seconds. In contrast, the dashed line corresponding to initiation of the departure process leads the solid departure line by \( d_a / 2 \) seconds.

Acceleration-deceleration delay \( d_a \) is split into its separate components of acceleration delay and deceleration delay. It is assumed that each component is one-half of the total (i.e., \( d_a / 2 \)). This assumption is made for convenience in developing the polygon. The derivation of the stop rate equations indicates that the two components are always combined as \( d_a \). Thus, the accuracy of the assumed distribution of delay to the two components does not influence the accuracy of the estimated stop rate.

The number of fully stopped vehicles \( N_f \) represents the number of vehicles that arrive before the queue of stopped vehicles has departed. By inspection of Figure E-2, the equation for computing this variable can be derived as:

\[
N_f = q_r r + q_g (t_f - d_a)
\]  

A second equation for estimating \( N_f \) can also be derived by inspection of Figure E-2 as follows:
Combining Equations 12 and 11 to eliminate Nf and solve for tf yields:

\[ t_f = \frac{q_r r - q_g d_a}{s - q_g} \]  

(13)

Equation 13 can be combined with Equation 11 (or 12) to obtain an estimate of Nf. The stop rate is then computed as:

\[ h = \frac{N_f}{\min(1, \lambda) e s} + \frac{N_o}{q C} \]  

(14)

If the relationships for qr and qs in the variable list associated with Figure E-2 are substituted in Equation 13, the combination of Equations 11, 13, and 14 reduces to Equation 2 (combined with Equation 3), as developed by Olszewski (3). As noted in the discussion of Olszewski’s model, two unique cases exist depending on whether da is less than “(1 - P) g X”. Equation 11 applies to the case where da is less than “(1 - P) g X”.

The derivation of stop rate equations for other combinations of lane allocation, left-turn operation mode, and phase sequence is similar to that for Equations 11 and 12. However, the number of equations increases significantly due to the associated complexity of these combinations and the number of unique cases that may exist.

DETERMINING STOP RATE

This part of the paper describes a procedure for determining the full stop rate at an intersection that is operating as part of a coordinated actuated signal system. The procedure is described in a narrative format and does not define every equation needed to compute stop rate for every combination of lane allocation, left-turn operational mode, and phase sequence. This approach is taken because of the large number of equations required to address the full range of combinations found at intersections in most cities. Nevertheless, all of these equations have been developed and are automated in the computational engine which is available from TRB’s Highway Capacity and Quality of Service Committee. Some of the equations presented in the previous part are repeated in this part for reader convenience.

Data Requirements

This section describes the data needed to use the procedure for determining the full stop rate for an intersection lane group. The lane group is served either by an actuated, non-coordinated phase or the coordinated phase at a coordinated actuated intersection. The data noted in this section represent the additional data that are needed, relative to that currently required to use the procedure in Chapter 16 of the HCM and that needed to estimate average phase duration (as described in Appendix D).
The additional data items needed for the calculation of stop rate are:

- Threshold Speed
- Deceleration Rate
- Acceleration Rate

The threshold speed represents the speed at or below which a vehicle is said to be effectively stopped while in queue or when joining a queue. The strictest definition of this speed is 0.0 mph, which coincides with a complete stop. However, vehicles sometimes move up in queue while drivers wait for the green indication. A vehicle that moves up in queue and then stops again does not incur an additional full stop. The threshold speed that is judged to differentiate between vehicles that truly stop and those that are just moving up in queue is 5 mph.

A deceleration rate and an acceleration rate are needed to compute the incremental delay associated with a speed change dictated by the traffic signal. If local values are not available, the deceleration rate can be estimated as 4.0 ft/s² and the acceleration rate can be estimated as 3.5 ft/s². These rates represent average values for all stopping vehicles. It is recognized that vehicles near the stop line at the onset of yellow may decelerate at twice the average rate. Similarly, the first vehicle in queue at the start of green may accelerate at twice the average value.

**Arrival-Departure Polygon**

This section describes the calculations needed to construct the arrival-departure polygon (ADP) associated with each lane group served during a phase. This diagram defines the cumulative number of arrivals and departures associated with a traffic movement as a function of time during the average cycle. The ADP associated with a lane group is a useful tool for deriving an equation for computing the number of full stops \(N_f\) incurred by vehicles in the lane group.

The shape of the ADP is defined by the arrival flow rate during the effective red and green intervals, saturation flow rate associated with each movement in the lane group, signal indication status, left-turn operation mode, and phase sequence. Once constructed, the polygon can be used to compute the queue service time, service time for fully stopped vehicles, and stop rate for the corresponding lane group.

The ADP for a through movement is shown in Figure E-3. It shows the polygon for the average cycle. The red and green intervals are ordered from left to right in the sequence of presentation, such that the last two time periods correspond to the queue service time \(g_s\) and green extension time \(g_e\) of the subject phase. The variables shown in the figure are defined as follows:

\[
\begin{align*}
g &= \text{effective green time, s;} \\
r &= \text{effective red time, s;} \\
t_f &= \text{service time for fully stopped vehicles, s;} \\
N_f &= \text{number of fully stopped vehicles (platooned arrivals), veh;} \\
d_a &= \text{deceleration-acceleration delay, s;} \\
g_s &= \text{queue service time (= } Q_r/[s - q_g]), \text{ s;} \\
\end{align*}
\]
\( g_e \) = green extension time, s;
\( q \) = average flow rate of traffic stream, veh/s;
\( q_L \) = lane group flow rate per lane (= \( q/N \)), veh/s/ln;
\( q_r \) = flow rate during the effective red time (= \( [1 - P] \frac{q_L C}{r} \)), veh/s/ln;
\( q_g \) = flow rate during the effective green time (= \( P \frac{q_L C}{g} \)), veh/s/ln;
\( Q_r \) = queue size at the end of effective red time (= \( q_r r \)), veh/s/ln;
\( N \) = number of lanes in lane group;
\( P \) = proportion of vehicles arriving on green (= \( R_p g/C \));
\( C \) = cycle length, s; and
\( s \) = saturation flow rate; veh/s/ln.

If the lane group flow rate \( q_L \) exceeds the lane capacity (= \( C/[s g] \)), then it is set equal to the lane capacity.

![Figure E-3. Arrival-departure polygon for protected movements.](image)

The higher solid trend line in Figure E-3 corresponds to vehicles arriving to the intersection. The lower solid trend line corresponds to queued vehicles departing the stop line. The lower line is horizontal during the effective red, denoting no departures. The vertical distance between these two lines represents the queue length. The dashed line in this figure represents only those vehicles that complete a full stop. The dashed line corresponding to arrivals to the stopped queue lags behind the solid arrival line by \( d_s/2 \) seconds. In contrast, the dashed line corresponding to the initiation of the departure process leads the solid departure line by \( d_s/2 \) seconds. For this polygon, and subsequent polygons shown, the procedure described in of Chapter 16 of the HCM is used to estimate the saturation flow rate \( s \).

The number of fully stopped vehicles \( N_f \) represents the number of vehicles that arrive before the queue of stopped vehicles has departed. The relationships in Figure E-3 can be used to derive an equation for estimating this number.

The polygon in Figure E-3 applies to through lane groups or left-turn lane groups served during a left-turn phase that operates in the protected-only mode. This polygon is also applicable
to split phasing. For split phasing, each approach is evaluated separately to determine its overall stop rate. If the approach has a left-turn lane, then a separate polygon is constructed for both the left-turn and the through lane group.

More complicated combinations of phase sequence and left-turn operational mode dictate more complicated arrival-departure polygons. A polygon must be uniquely derived for each combination. The most common combinations are illustrated in Figures E-4, E-5, E-6, and E-7. The variables shown in these figures are defined in the previous list, with the following additions:

\[ g_p = \text{effective green time for permissive left-turn operation, s}; \]
\[ g_u = \text{duration of } g_p \text{ that is not blocked by an opposing queue, s}; \]
\[ g_f = \text{time before the first left-turn vehicle arrives and blocks the shared lane, s}; \]
\[ s_p = \text{saturation flow rate of permissive left-turn operation, veh/s/ln}; \]
\[ p_x = \text{probability of phase termination by reaching the maximum green limit}; \] and
\[ P_L = \text{proportion of left-turn vehicles in the inside lane of a shared lane approach}. \]

![Figure E-4. Arrival-departure polygon for permissive-only left turns from exclusive lane.](image)

![Figure E-5. Arrival-departure polygon for permissive-only left turns from shared lane.](image)
The polygon in Figure E-4 applies to the left-turn lane group that operates as permissive only during the adjacent through phase. Figure E-3 applies to the adjacent through lane group. If the phase extends to max out such that $g_e$ is equal to 0.0, then some left-turns will be served as sneakers.

The polygon in Figure E-5 applies to shared lane approaches. If the approach has two or more lanes, the polygon in Figure E-5 applies to the inside lane serving left-turn and through vehicles. Figure E-3 is used for the outside lanes. If the phase extends to max out such that $g_e$ is equal to 0.0, then some left-turns will be served as sneakers. The expected number of sneakers is computed as $(1 + P_I)$, where $P_I$ is the proportion of left-turn vehicles in the inside lane.

The polygon in Figure E-6 applies to left-turn movements that have protected-permissive operation with a leading left-turn phase. The polygon in Figure E-7 applies to the same movements.
and operation but with a lagging left-turn phase. If a queue exists at the end of the permissive period for either polygon, then the queue is reduced by the number of sneakers.

The remaining discussion in this section describes the sequence of calculations needed to construct the arrival-departure polygon. This sequence may vary, depending on the left-turn operational mode, phase sequence, and lane assignments for the subject lane group. It is assumed that the queue accumulation polygon and associated green times (i.e., $g$, $g_p$, $g_f$, $g_u$, $g_s$, $g_e$) have been developed using the procedure described in Appendix D.

**A. Determine Acceleration-Deceleration Delay ($d_a$).** The deceleration-acceleration delay $d_a$ term is used to distinguish between a fully and a partially stopped vehicle. For typical speeds and rates, $d_a$ ranges from 8 s to 14 s. It can be computed using the following equation:

$$d_a = 0.735 \frac{(S_R - S_s)^2}{S_R} \left(\frac{1}{r_a} + \frac{1}{r_d}\right)$$

where,
- $S_R$ = running speed, mph;
- $S_s$ = threshold speed defining a stopped vehicle, mph;
- $r_a$ = acceleration rate, ft/s$^2$; and
- $r_d$ = deceleration rate, ft/s$^2$.

**B. Define Arrival-Departure Polygon.** During this step, the green times and flow rates used to construct the queue accumulation polygon are used to construct the arrival-departure polygon. The appropriate polygon shape is selected from Figures E-3 through E-7 based on the left-turn mode of operation, phase sequence, and lane assignments. The mode, sequence, and assignment should be the same as that used to guide the construction of the queue accumulation polygon.

**C. Define Arrival-Departure Polygon for Stopped Vehicles.** During this step, the polygon defined in the previous step is enhanced to include the polygon shape for the stopped vehicles. The stopped vehicle polygon is defined using dashed lines in Figures E-3 through E-7. Two rules guide the development of this polygon. First, the dashed line corresponding to arrivals to the stopped queue lags behind the solid arrival line by $d_a/2$ seconds. Second, the dashed line corresponding to initiation of the departure process leads the solid departure line by $d_a/2$ seconds.

**D. Compute Service Time for Fully Stopped Vehicles ($t_f$).** The service time is computed for each polygon constructed in the previous step. For through movements, or left-turn movements served during a left-turn phase operating in the protected-only mode, the polygon in Figure E-3 applies and the appropriate one of the following two equations can be used:

$$t_f = \frac{q_L C (1 - P - P d_a/g)}{s (1 - \text{min}[1,X]) P} : \text{if } \frac{d_a}{2} \leq (1 - P) g X$$

$$t_f = \frac{q_L C (1 - P) (r - d_a)}{s (r - [1 - P] g \text{min}[1,X])} : \text{if } \frac{d_a}{2} > (1 - P) g X$$
where, 
\[ X = \text{volume-to-capacity ratio } (= q_L C / [s g]). \]

The saturation flow rate \( s \) used in Equations 16 and 17 is obtained from Equation 16-4 of Chapter 16 of the HCM.

E. Compute the Number of Fully Stopped Vehicles for Platoon Arrival \((N_f)\). The number of fully stopped vehicles for platooned arrivals is computed for each polygon constructed in Step C. For through movements, or left-turn movements served during a left-turn phase operating in the protected-only mode, the polygon in Figure E-3 applies and the appropriate one of the following two equations can be used:

\[
N_f = q_r r + q_0 (t_f - d_a) \quad : \text{If} \ d_a < (1 - P) g X \quad (18)
\]
\[
N_f = q_0 (r - d_a + t_f) \quad : \text{If} \ d_a > (1 - P) g X \quad (19)
\]

If the volume-to-capacity ratio in Equations 18 and 19 exceeds 1.0, then it is set equal to 1.0.

F. Compute the Number of Fully Stopped Vehicles During Overflow \((N_o)\). The number of fully stopped vehicles during overflow is computed for each lane group. The following equation can be used for lane groups served during an actuated phase:

\[
N_o = 0.25 c_a T \left[ \left( X_a - 1 + \frac{Q_b}{c_a T} \right) + \sqrt{\left( X_a - 1 + \frac{Q_b}{c_a T} \right)^2 + \frac{8 k_b X_a}{c_a T} + \frac{16 k_b Q_b}{(c_a T)^2}} \right] \quad (20)
\]

with,

\[
k_b = 0.10 I (s g)^{0.6} \quad (21)
\]
\[
c_a = 3600 \frac{g_a s N}{C} \quad (22)
\]
\[
g_a = G_{\text{max}} + Y - l_1 - l_2 \quad (23)
\]

where,
\[
N_o = \text{overflow queue for the lane group, veh;}
\]
\[
c_a = \text{lane group capacity available to an actuated phase, veh/h;}
\]
\[
T = \text{length of analysis period, h;}
\]
\[
X_a = \text{available volume-to-capacity ratio } (= 3600 q/c_a);
\]
\[
k_b = \text{second-term incremental factor;}
\]
\[
I = \text{upstream filtering factor for platooned arrivals (from Exhibit 15-7 of the HCM);}
\]
\[
Q_b = \text{initial queue at the start of the analysis period, veh;}
\]
\[
G = \text{average green interval duration, s;}
\]
\[
G_{\text{max}} = \text{maximum green setting for the subject phase, s;}
\]
\[
l_1 = \text{start-up lost time, s;}
\]
\[
l_2 = \text{end lost time, s } (= Y - 2.0) \text{; and}
\]
\[
Y = \text{combined yellow change and red clearance interval duration (intergreen), s.}
\]
If the lane group is served during the coordinated phase, then the following equation is used:

\[ N_o = 0.25 c T \left( X - 1 + \frac{Q_b}{c T} \right) + \sqrt{ \left( X - 1 + \frac{Q_b}{c T} \right)^2 + \frac{8 k_b X}{c T} + \frac{16 k_b Q_b}{(c T)^2} } \]  

(24)

with,

\[ k_b = 0.12 I (s g)^{0.7} \]  

(25)

\[ c = 3600 \frac{g s N}{C} \]  

(26)

where,

\[ c = \text{ lane group capacity, veh/h.} \]

**G. Compute the Full Stop Rate (h).** The number of fully stopped vehicles for a lane group is computed using the following equation:

\[ h = \frac{N_f}{\min(1, X) g s} + \frac{N_o}{q C} \]  

(27)

where,

\[ h = \text{ full stop rate, stops/veh.} \]

If the full stop rate is multiplied by 100, the product can be interpreted as the percentage of vehicles stopping.

**SENSITIVITY ANALYSIS**

This part of the appendix examines the sensitivity of full stop rate to variation in volume-to-capacity ratio and progression quality. Equations 16 through 23 and 27 were used for this analysis.

Figure E-8 shows the effect of volume-to-capacity ratio on the two stop rate components: stops due to platooned arrivals \( N_p \) and stops due to overflow \( N_o \) (i.e., cycle failure). The stops due to platooned arrivals are shown with a dashed line. The stops due to overflow are shown with a solid line. Both trend lines increase with increasing volume-to-capacity ratio. The stops due to platooned arrivals plateau at about 0.83 stops/veh when the volume-to-capacity ratio exceeds 1.0. This limiting value is consistent with the 0.9 stops/veh obtained from the first term of Equation 1. The number of stops due to cycle failure is negligible for a volume-to-capacity ratio less than 0.8. However, the number of secondary stops increases significantly as this ratio exceeds 0.8.

Figure E-9 illustrates the effect of progression quality and volume-to-capacity ratio on the full stop rate. The trend lines shown in this figure represent the sum of: (1) the stops due to platooned arrivals, and (2) the stops due to overflow. The three trend lines shown indicate the effect of different degrees of progression quality, as quantified by the proportion of vehicles arriving on green \( P \). The effect of progression quality is shown to be more significant with lower volume-to-capacity ratios. Poor progression tends to significantly increase the full stop rate for ratios less than 0.8. It is noted that even with good progression and a moderate volume-to-capacity ratio, 40 to 60 percent of vehicles still incur a full stop (i.e., the stop rate ranges from 0.4 to 0.6 stops/veh).
SELECTED MODEL CALIBRATION

This part of the appendix summarizes the findings from the calibration of the stop rate prediction procedure. The calibration was based on a comparison of the computed stop rate with the stop rate obtained from the CORSIM traffic simulation model (version 5.1).

The simulation testbed was based on the S.W. Barbur Boulevard street segment located between S.W. 19th Avenue and S.W. Bertha Boulevard. This segment is 2937 ft in length, has two
through lanes in each direction, and a speed limit of 35 mph. The left-turn movements for the coordinated street at both intersections were operated using the protected mode and led the through phases. The left-turn movements on S.W. 19th Avenue were operated using the protected-permissive mode and led the through phases. Split phasing was used on S.W. Bertha Boulevard. The cycle length was 100 s. Progression was established for the peak travel direction and was characterized as Arrival Type 4. Progression for the off-peak direction was characterized as Arrival Type 3.

Sixteen volume scenarios were evaluated. The procedure was used to estimate the average green interval duration for each phase at each intersection. A total of 224 averages were obtained in this manner (= 8 phases x 16 volume combinations at one intersection + 6 phases x 16 volume combinations at the other intersection).

The full stop rate for each phase at each intersection was recorded each cycle using post-processing software. The post-processor scanned the animation file produced by CORSIM and extracted the desired data. The animation file contains a second-by-second record of signal status for each simulation run. A vehicle was defined as having stopped if its speed was less than 5 mph. Secondary stops were included whenever the vehicle accelerated to a speed above 5 mph and then decelerated to a speed below 5 mph. This threshold speed was reasoned to minimize the count of secondary stops due to queue consolidation but to accurately count those stops due to overflow (i.e., cycle failure). A total of 224 stop rate estimates were obtained in this manner (= 8 phases x 16 hours at one intersection + 6 phases x 16 hours at the other intersection).

Figure E-10 compares the computed full stop rate with the full stop rate obtained from the simulation runs. Each data point represents one hour of simulation for one phase at one intersection. The thick black trend line represents the best-fit regression line. The equation for this line, and its coefficient of determination $R^2$, are shown in the figure. The slope coefficient is not significantly different from 1.0 and the intercept coefficient is not significantly different from 0.0 s. These findings lead to the conclusion that the procedure can be used to estimate the full stop rate without bias and that it does not need additional calibration.

Figure E-10. Comparison of computed and simulated full stop rate.
The root mean square error for the regression line is 0.056 stops/veh. This variable provides an indication of the standard deviation of the prediction. It is based on one hour of simulation for each data point. Smaller values are likely for longer simulation runs.

REFERENCES


APPENDIX F

Procedure for Addressing Capacity Constraints
APPENDIX F

PROCEDURE FOR ADDRESSING CAPACITY CONSTRAINTS

INTRODUCTION

This appendix describes a procedure for addressing the effect of two capacity-related factors that affect the steady flow of traffic along an urban street. The first factor is described as queue spillback. Spillback describes the situation where a queue has grown backward along a segment to the extent that it reaches the upstream signalized intersection. The second factor is described as upstream metering. Metering occurs when the capacity of the upstream movements limit the number of vehicles that can reach the subject downstream intersection.

The procedure described in this appendix is used for two purposes. First, it is used to adjust traffic movement volumes on the major street approaches to account for upstream metering. Then, it is used to identify if spillback will occur during the analysis period and, if it does occur, when it occurs.

The procedures described in previous appendices do not address traffic flow during spillback conditions. As such, the accuracy of the methodology being developed for Chapter 15 of the Highway Capacity Manual (HCM) will degrade when spillback occurs. This limitation also exists with the methodology in Chapter 15 of the current edition of the HCM (1).

This appendix consists of three parts. The first part provides a review of literature related to upstream metering and queue spillback. The second part describes the procedure for determining stop rate. The last part describes the procedure in an example application.

BACKGROUND

This part of the appendix provides a brief review of the literature on the topics of upstream metering and queue spillback. Each topic is addressed in a separate section. The third section notes some queuing related factors that can also have an effect on intersection capacity and operation.

Upstream Metering

It was noted by Tarko (2) that the planning and preliminary design applications, for which the HCM procedures are sometimes used, are often based on forecast or estimated volumes. When forecast volumes are used, they may overestimate the actual number of vehicles that can travel down the street when its capacity is considered. When the forecast volume for an intersection traffic movement exceeds its capacity, the volume arriving to a downstream intersection is limited (or metered) by the capacity of the upstream movement. When metering occurs, the volume arriving at a downstream signal may not equal the volume obtained from a forecast or estimation technique. Metering can also occur during the evaluation of alternative conditions for an existing street system.
The impact of metering on travel speed along an urban street facility is shown in Figure F-1. The trend line labeled “Current” represents the travel speed predicted by the HCM procedure. The input volume referenced in this figure is the through lane-group volume at the first intersection of the facility. The trends shown in the figure indicate that the travel speed estimate from the HCM procedure can be 5 to 10 mph slower than that the actual travel speed when metering is present.

![Figure F-1](image)

*Figure F-1. Effect of upstream signal metering on travel speed. (2)*

**Queue Spillback**

Spillback can be characterized as one of two types: cyclic and sustained. Cyclic spillback can occur each signal cycle and is a result of queue growth during the red indication. When the green indication is presented, the queue dissipates and spillback is no longer present for the remainder of the cycle. This type of spillback can occur on short street segments with relatively long signal cycle lengths.

Sustained spillback occurs at some point during the analysis period and is a result of oversaturation (i.e., more vehicles discharging from the upstream intersection than can be served at the subject downstream intersection). The queue does not dissipate at the end of each cycle. Rather, it remains present until the downstream capacity is increased or the upstream demand is reduced.

**Cyclic Spillback**

The impact of cyclic spillback on delay is illustrated in Figure F-2a. This figure shows the cumulative number of arrivals and departures at the subject downstream intersection during one signal cycle. During the red indication, the queue (represented as the vertical distance between the arrival and discharge trend lines) is shown to grow until it reaches the segment length. At this point, no additional vehicles can be stored on the segment. The delay incurred by the vehicles on the...
subject segment is indicated by the area bounded by the solid lines. It excludes the triangular shaped region bounded on the top by a dashed line and on the bottom by a solid line.

\[ Q_0 \]

Delay at subject intersection inside area bounded by solid lines.

\[ \text{Cyclic Spillback} \]

Delay at subject intersection inside area bounded by solid lines.

\[ \text{Sustained Spillback} \]

Sustained spillback occurs at some point during the analysis period and is a result of oversaturation at the downstream intersection. The impact of sustained spillback on delay is illustrated in Figure F-2b. This figure shows the cumulative number of arrivals and departures for several signal cycles.

During the red indication of the second cycle, the cumulative number of arrivals exceeds the cumulative number of discharging vehicles. At this point, no additional vehicles can be stored on the segment. The delay incurred by the vehicles on the subject segment is indicated by the area bounded by the solid lines. It excludes the area bounded on the top by a dashed line and on the bottom by a solid line. The arrivals that cannot store on the subject segment are stored on the side street or on upstream approaches.

Sustained spillback can occur on segments of any length. Prior to the occurrence of spillback, the procedures described in the *HCM* can be used with reasonable accuracy. However,
after it occurs, the assumptions inherent to the HCM methodology are not valid. Simulation models may be more appropriate for modeling facility operation when spillback exists.

Other Constraints

In addition to queue spillback, there is another source of capacity constraint that occurs at closely-spaced signalized intersections. This constraint is imposed on one or more of the signal phases at the downstream intersection. It occurs when the subject phase is green but the upstream signal phasing is such that no vehicles have been discharged in time to take advantage of the green indication. This inability to efficiently use the downstream green indication is termed “demand starvation.” It is most significant for short segment lengths with flows entering the segment from only one or two signalized movements at an upstream intersection, such as a signalized interchange.

Spillback can also affect unsignalized intersection capacity. Specifically, the presence of a queue from a downstream signalized intersection can impede service to one or more non-priority movements at a two-way stop-controlled intersection. The time duration of queue spillback at an unsignalized intersection can be estimated from the queue accumulation polygon derived for each lane group at the downstream signalized intersection.

Models that account for the effect of demand starvation and spillback-related impedance are beyond the scope of this research. They are recommended as the subject of future research.

DETERMINING CAPACITY CONSTRAINTS

This part of the appendix describes a procedure for addressing the effect of capacity constraints on urban street operation. The procedure has two components. The first component is used to adjust traffic movement volumes on the major street approaches to account for upstream metering. The second component is used to identify if spillback will occur during the analysis period and, if it does occur, when it occurs. If spillback occurs, the analysis period must not exceed the time before spillback. This requirement specifically affects the value used for the variable \( T \) in the incremental delay \( d_2 \) equation, average overflow queue per lane equation \( Q_2 \), and the overflow queue for the lane group equation \( N_o \). The first two of these equations are described in Chapter 16 of the HCM. The third equation is described in Appendix E.

The procedure is based on an analysis of overall flow conditions during the analysis period. This period typically ranges from 0.25 to 1.0 h. The procedure must be implemented prior to the estimation of average phase duration, delay, or stop rate. It is most appropriate when the volumes are forecast or during alternative analysis where the alternatives considered have an effect on the capacity of one or more movements. It does not need to be applied when evaluating an existing urban street for which the volumes were measured in the field. The procedure does not address cyclic spillback, demand starvation, or spillback-related impedance at an unsignalized intersection.
Volume Check and Metering Analysis

This section describes the procedure for determining the turn movement volumes at each intersection along the subject urban street. The analysis is separately applied to each travel direction and proceeds in the direction of travel. The procedure consists of a series of steps that are completed in sequence for the “entry” and “exit” movements associated with each segment. These movements are shown in Figure F-3.

As indicated in Figure F-3, there are three entry movements associated with the upstream signalized intersection and three exit movements associated with the downstream signalized intersection. There are also entry and exit movements at each mid-segment access point. However, these movements are aggregated into one entry and one exit volume for simplicity. The analysis steps are described in the following paragraphs.

A. Identify Arrival Volume. During this step, the arrival volume for each entry movement is identified. The volumes may be obtained from forecast estimates or derived from average annual daily traffic counts.

The volume entering the segment from each access point should be identified and added to obtain a total for the segment. Similarly, the volume exiting the segment from each access point should be added for the segment.

B. Estimate Phase Capacity \((c)\). During this step, the capacity of each signalized entry or exit movement is estimated. This estimate should be a reasonable approximation based on estimates of the saturation flow rate for the corresponding movement and the phase splits established for signal coordination. The following equation can be used to estimate phase capacity:

\[
c = \frac{g_s N}{C}
\]  

(1)
where,
\[ c = \text{capacity, veh/h.} \]
\[ g = \text{effective green time, s;} \]
\[ N = \text{number of lanes;} \]
\[ C = \text{cycle length, s; and} \]
\[ s = \text{saturation flow rate; veh/h/ln.} \]

For movements served in an exclusive lane, a saturation flow rate of 1800 veh/h/ln can be used in Equation 1. If the cross-street turn movement shares a lane with its adjacent through movement, then the discharge flow rate for the turn movement can be estimated using the following equation:

\[ s_{q|u} = \frac{1800 \ P_T}{1 + P_T (E_T - 1)} \]

with,
\[ P_T = P_t \ n_s \leq 1.0 \]

where,
\[ s_{q|u} = \text{shared lane discharge flow rate for upstream traffic movement } u, \text{ veh/h/ln;} \]
\[ P_T = \text{proportion of turning vehicles in the shared lane;} \]
\[ P_t = \text{proportion of turning vehicles in the through lane group that shares a lane with the subject turn movement;} \]
\[ n_s = \text{number of lanes in the through lane group that shares a lane with the subject turn movement;} \]
\[ E_T = \text{through-car equivalent for the turn movement.} \]

When Equation 2 is applied to a left-turn movement, the through-car equivalent variable can be obtained from Exhibit C16-3 in Chapter 16 of the *HCM*. This exhibit indicates that \( E_T \) equals 1.4, 2.5, and 4.5, for opposing flows of 1, 600, and 1200 veh/h, respectively. When Equation 2 is applied to a right-turn movement, \( E_T \) equals 1.18.

**C. Compute Volume-to-Capacity Ratio (X).** During this step, the volume-to-capacity ratio is computed for each signalized entry movement. This ratio is computed by dividing the arrival volume from Step A by the capacity estimated in Step B. Any movements with a volume-to-capacity ratio in excess of 1.0 will meter the volume arriving to the downstream intersection.

**D. Compute Discharge Volume (v_d).** The volume discharged from each signalized entry movement is equal to the smaller of the arrival volume or the associated movement capacity. The mid-segment discharge volume is equal to its arrival volume. The sum of the four discharge volumes equals the total discharge volume. It represents all of the vehicles that can arrive to the downstream intersection, less those that exit at a mid-segment access point.

**E. Compute Adjusted Arrival Volume (v_a).** The total discharge volume from Step D should be compared with the sum of the four exiting volumes established in Step A for the subject segment. If the two totals do not agree in magnitude, the four exiting volumes must be adjusted such
that their sum equals the total discharge volume. Unless specific knowledge is available about the nature of the adjustments, it is recommended that each exit volume be multiplied by the ratio of the total discharge volume to the total exiting volume.

**F. Repeat Steps A Through E for Each Segment.** The steps above should be completed for each segment in the facility in the subject direction of travel. The procedure should then be repeated for the opposing direction of travel.

**Spillback Analysis**

This section describes the procedure for determining if queue spillback occurs on one or more segments of an urban street facility. The analysis is separately applied to each travel direction and proceeds in the direction of travel. The procedure consists of a series of steps that are completed in sequence for the signalized exit movements associated with each segment. These movements are shown in Figure F-3. Spillback due to the movements associated with the access points is not specifically addressed.

**A. Identify Initial Queue (Q₀).** During this step, the initial queue for each signalized exit movement is identified. This value represents the queue present at the start of the analysis period (a total of all vehicles in all lanes serving the movement). The initial queue estimate would likely be available for the evaluation of an existing condition where field observations indicate the presence of a queue at the start of the analysis period. For planning or preliminary design applications, it can be assumed to equal 0.0 veh.

**B. Identify Queue Storage Length (Lₚᵢ).** The length of queue storage for each exit movement is identified during this step. For turn movements served from a turn bay, this length equals the length of the turn bay. For through movements, this length equals the segment length less the width of the upstream intersection. For turn movements served from a lane equal in length to that of the segment, the queue storage length equals the segment length less the width of the upstream intersection.

**C. Compute Maximum Queue Storage (Nₚᵢ).** The maximum queue storage for each exit movement is computed using the following equation:

\[
N_{ₚᵢ} = \frac{N \cdot L_{ₚᵢ}}{L_{ₚᵢ}}
\]

where,

- \(N_{ₚᵢ}\) = maximum queue storage, veh;
- \(L_{ₚᵢ}\) = length of roadway between the front of two queued vehicles (= 25 ft/veh), ft/veh; and
- \(L_{ₚᵢ}\) = queue storage length, ft.

**D. Compute Available Storage Length (Nₐₚᵢ).** The available storage length is computed for each signalized exit movement using the following equation:

\[
N_{ₚᵢ} = N_{ₚᵢ} - Q₀ \geq 0.0
\]
where,
\[ N_{qa} = \text{available storage length, veh}; \text{ and} \]
\[ Q_o = \text{initial queue length, veh}. \]

The analysis thus far has focused on the three signalized exit movements. At this point, the analysis must be extended to include the combined through and left-turn movement when the left-turn movement has a bay (i.e., it does not have a lane that extends the length of the segment). The analysis must also be extended to include the combined through and right-turn movement when the right-turn movement has a bay (but not a full-length lane).

The analysis of these newly formed “combined movements” is separated into two parts. The first part represents the analysis of just the bay. This analysis represents a continuation of the exit movement analysis using the subsequent steps of this procedure. The second part represents the analysis of the length of segment shared by the turn movement and the adjacent through movement. The following rules are used to evaluate the combined movements for the shared segment length:

1. The volume for each combined movement equals the sum of the adjusted arrival volume for the two contributing movements.
2. The initial queue for each combined movement is computed using the following equation:
   \[ Q_o = \text{larger of:} \left\{ 0.0, \quad Q_{o,\text{turn}} - \frac{L_{qo,\text{turn}}N_{turn}}{L_{qo}}, \quad Q_{o,\text{thru}} - \frac{L_{qo,\text{thru}}N_{thru}}{L_{qo}} \right\} \]  
   \[ (6) \]
3. The queue storage length for each combined movement equals the queue storage length for the through movement less the queue storage length of the turn movement.
4. The lanes available to the combined movement equals the number of lanes available to the through movement.
5. The maximum queue storage for the combined movement is computed using the guidance provided in Step C for the exit movements.
6. The available storage length for the combined movement is computed using the guidance provided previously in this step for the exit movements.

E. Compute Capacity \( (c) \).

During this step the capacity for the exit movements and the combined movements is computed. In fact, the capacity for each exit movement was computed in Step B in the previous section. The capacity of the combined movements is computed using the following equation:

\[ c = \frac{v_{a,1}}{X_1} + \frac{c_{\text{thru}} (N_{\text{thru}} - 1)}{N_{\text{thru}}} \]  
   \[ (7) \]

with

\[ v_{a,1} = \text{larger of:} \left\{ v_{a,\text{turn}}, \quad \frac{v_{a,\text{turn}} + v_{a,\text{thru}}}{N_{\text{thru}}} \right\} \]  
   \[ (8) \]
\[ X_1 = \frac{v_{a,\text{turn}}}{c_{\text{turn}}} + \frac{v_{a,1} - v_{a,\text{turn}}}{c_{\text{thru}}/N_{\text{thru}}} \]  
   \[ (9) \]
where,
\[ v_{a,1} = \text{adjusted arrival volume in the shared lane, veh/h;} \]
\[ X_1 = \text{volume-to-capacity ratio in the shared lane;} \]
\[ c_{\text{thru}} = \text{capacity for the through exit movement, veh/h;} \]
\[ c_{\text{turn}} = \text{capacity for the subject turn exit movement, veh/h;} \]
\[ N_{\text{thru}} = \text{number of lanes serving the through exit movement;} \]
\[ v_{a,\text{turn}} = \text{adjusted arrival volume for the subject turn movement, veh/h; and} \]
\[ v_{a,\text{thru}} = \text{adjusted arrival volume for the through movement, veh/h.} \]

**F. Compute Queue Growth Rate \((rqg)\).** During this step, the queue growth rate is computed for each signalized exit movement for which the storage extends the length of the segment. Typically, the through movement satisfies this requirement. A turn movement may also satisfy this requirement if it is served by an exclusive lane that extends the length of the segment. The queue growth rate is computed as the difference between the adjusted arrival volume \(v_a\) and the capacity \(c\) for the subject exit movement. The discharge rate for each exit movement was computed in Step E in the previous section. The equation for computing this rate is:

\[ rqg = v_a - c \geq 0.0 \]  \hspace{1cm} (10)

where,
\[ rqg = \text{queue growth rate, veh/h;} \]
\[ v_a = \text{adjusted arrival volume, veh/h; and} \]
\[ c = \text{capacity, veh/h.} \]

The queue growth rate is also computed for the combined movements formulated in Step D. The adjusted volume used in Equation 10 would represent the sum of the through and turn movement volumes represented in the combined group. The capacity for the group was computed in Step E.

**G. Compute Time Until Spillback \((T_c)\).** During this step, the time until spillback is computed for each signalized exit movement for which the storage extends the length of the segment. This time is computed using the following equation for any movement with a nonzero queue growth rate:

\[ T_c = \frac{N_{\text{qq}}}{rqg} \]  \hspace{1cm} (11)

where,
\[ T_c = \text{time until spillback, h.} \]

For turn movements served by a bay, the computed spillback time represents the time required for the bay to overflow. It does not represent the time that the turn-related queue reaches the upstream intersection.

Equation 11 is also used to compute the spillback time for the combined movements formulated in Step D. However, this time represents the additional time required for the queue to grow along the length of segment shared by the turn movement and the adjacent through movement.
This additional time must be added to the time required for the corresponding turn movement to overflow its bay to obtain the spillback time for the combined movement.

H. Repeat Steps A Through G for Each Segment. The preceding steps should be completed for each segment in the facility in the subject direction of travel. The procedure should then be repeated for the opposing direction of travel.

I. Determine the Controlling Spillback Time. During this step, the shortest time until spillback for each of the exit movements (or movement groups) for each segment and direction of travel is identified. If the segment supports two travel directions, then two values are identified (one value for each direction). The shorter value of the two represents the controlling spillback time for the segment. If a movement (or movement group) does not spill back, it is not considered in this process for determining the controlling spillback time.

Next, the controlling segment times are compared for all segments that comprise the facility. The shortest time found represents the controlling spillback time for the facility.

If no one movement associated with the urban street facility experiences spillback, or if the spillback time exceeds the desired analysis period, then the HCM evaluation procedure can be used to evaluate the facility. If spillback occurs before the desired analysis period ends, the duration of this period must be reduced such that it does not exceed the time before spillback occurs. The HCM procedure is still used for the evaluation. However, the adjusted analysis period is used for the variable $T$ in the incremental delay $d_2$ equation, average overflow queue per lane equation $Q_2$, and the overflow queue for the lane group equation $N_o$. The first two of these equations are described in Chapter 16 of the HCM. The third equation is described in Appendix E.

EXAMPLE APPLICATION

The procedure for addressing capacity constraints is described in this part of the appendix using an example application. Consider the two adjacent street segments shown in Figure F-4. Intersection B is common to both segments. Each intersection has movement a volume $v$, capacity $c$, and initial queue $Q_o$ shown. Each segment is 1320 ft in length and has two through lanes in the subject direction of travel (i.e., left to right). The desired analysis period $T$ is one hour. The capacity values shown are computed based on known phase splits, cycle length, saturation flow rate, and number of lanes.

The metering analysis proceeds, as shown in Tables F-1 and F-2, with the computation of capacity for each of the entry and exit movements. It turns out that the through movement volume arriving to each segment is metered by its capacity of 1000 veh/h. The volume discharging from both intersection A and intersection B is 1350 veh/h ($= 200 + 1000 + 100 + 50$). The forecast volume for intersection B, as shown in Table F-1, indicates a volume of 1485 veh/h, which exceeds the 1350 veh/h discharging from the upstream sources. These forecast volumes are each multiplied by the ratio 0.91 ($= 1350/1485$) to obtain the adjusted arrival volumes in the last row of Table F-1. A similar process is used to obtain the adjusted arrival volumes in the last row of Table F-2.
a. Segment from intersection A to intersection B.

b. Segment from intersection B to intersection C.

Figure F-4. Example street segment to illustrate metering and spillback effects.
TABLE F-1  Procedure for estimating metered volumes on segment A to B

<table>
<thead>
<tr>
<th>Variable</th>
<th>Upstream Intersection</th>
<th>Downstream Intersection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cross Street Left</td>
<td>Major Street Through</td>
</tr>
<tr>
<td>Arrival volume(^1), veh/h</td>
<td>200</td>
<td>1500</td>
</tr>
<tr>
<td>Phase capacity(^1) (= g/C × s × N)</td>
<td>500</td>
<td>1000</td>
</tr>
<tr>
<td>Volume-to-capacity ratio</td>
<td>0.40</td>
<td>1.50</td>
</tr>
<tr>
<td>Discharge volume, veh/h</td>
<td>200</td>
<td>1000</td>
</tr>
</tbody>
</table>

Forecast volume\(^1\), veh/h    | 55                    | 1331                    | 44                   | 55                 | 1485  |
 Adjusted arrival volume, veh/h   | 50                    | 1210                    | 40                   | 50                 | 1350  |

Note:
1 - Underlined values are inputs, all other values are computed from input values.

The spillback analysis proceeds, as shown in Tables F-3 and F-4, with the calculation of available storage length. When empty, each segment has storage for 106 vehicles (= 2 ×1320/25) based on an assumed storage length of 25 ft/veh. Given that there are 10 through vehicles stored on segment A to B at the start of the analysis period, the storage remaining will accommodate 96 through vehicles (= 106 - 10). The through movement at the downstream intersection of both segments has a capacity of 1000 veh/h. On segment A to B, a queue grows backward at a rate of 210 veh/h (= 1210 - 1000). Spillback on this segment by the through movement into the upstream intersection occurs 0.46 hours (= 96 / 210) after the start of the analysis period.
The queue growth rate computed for the left- and right-turn movements on either segment represents the time of bay overflow because these movements are served by a bay. As shown in the last row of Table F-4, the left-turn bay on segment B to C overflows into the adjacent through lane in 0.10 h.

The question of whether bay overflow contributes to spillback is answered by the analysis of the combined movements. This analysis is shown in the last two columns of Tables F-3 and F-4. The combined movement analysis considers the queue growth rate for the combined movements, relative to their combined storage space. The left-turn movement at intersection C (shown in Table F-4) is the only turn movement to overflow its bay and, thus, is the only combined movement with the potential to contribute to spillback (although the calculations were completed for all combined movements).
The combined adjusted volume for the left+through group on segment B to C is 1200 veh/h ($= 1100 + 100$). The queue storage length of 1120 ft represents that portion of the segment measured from the start of the bay to the upstream intersection ($= 1320 - 200$). This “partial” segment is shared by the left-turn and through movements. The available storage of 90 veh is based on the 1120 ft of storage and two lanes ($= 1120/25 \times 2$). The capacity of 784 veh/h represents the combined capacity for the two lanes and is less than 1000 veh/h because of the adverse interaction of the two movements. The queue growth rate within the partial segment is 416 veh/h ($= 1200 - 784$). The partial segment fills in 0.22 h ($= 90 / 416$); however, it takes the first 0.10 h for the left-turn bay to overflow. Thus, the combined group spills back in 0.32 h ($= 0.22 + 0.10$). Since this value is smaller than that for any movement that extends the length of the segment, it represents the critical time until spillback for this segment and direction of travel.

The queue growth rate for several movements in Tables F-3 and F-4 is zero. This implies that bay overflow or spillback will not occur as a result of the corresponding movements (although cyclic spillback may still occur). The time until spillback for these movements is identified as “> 1.0 h” because they do not impact the accuracy of the evaluation. A value of 1.0 h is used here because it represents the desired analysis period, as stated at the start of this example application.

Of the two segments used in this example application, the critical time for segment A to B is 0.46 h and the critical time for segment B to C is 0.32 h. Of the two values, 0.32 h is the shortest and represents the controlling time for the subject direction of travel on the two-segment facility. If a shorter time was found in the analysis of the other travel direction, then it would represent the controlling time until spillback.

REFERENCES


APPENDIX G

Verification of Methodology and Engine
APPENDIX G

VERIFICATION OF METHODOLOGY AND ENGINE

INTRODUCTION

This appendix describes the activities undertaken to verify the accuracy of the proposed urban streets methodology and associated software engine. The objective of this verification process is to demonstrate the ability of the methodology to accurately predict automobile performance for a wide range of conditions. A secondary objective of this process is to demonstrate that the computational engine has faithfully implemented the methodology.

The verification was based on a comparison of performance estimates from the engine with those obtained from a traffic simulation model. Four street segments were selected for this evaluation. They represent a subset of the six street segments for which data were collected during the full-segment field studies. The evaluation activities included the initial coding of a data file for each segment and the subsequent comparison of estimates from the engine and simulation model.

This appendix consists of three parts. The first part describes the approach used to evaluate the proposed methodology and engine. The second part discusses the findings from the evaluation. The last part summarizes the conclusions reached.

EVALUATION APPROACH

This part of the appendix describes the evaluation approach. The first section provides a description of the street segments used for the basis for the evaluation. The second section briefly reviews the process used to conduct the comparisons and identifies the performance measures used.

Study Site Description

Table G-1 shows the sites at which field data were collected for the full-segment study. Each site listed in the table is represented as one urban street segment. Additional details about these sites and the specific types of data collected were described previously in Appendix B.

<table>
<thead>
<tr>
<th>Field Study Technique</th>
<th>Segment</th>
<th>Location</th>
<th>Street Class</th>
<th>Segment Length, ft</th>
<th>Speed Limit, mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full Segment</td>
<td>Aviation Parkway</td>
<td>Tucson, Arizona</td>
<td>I</td>
<td>2800</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>S.E. McLoughlin Blvd.</td>
<td>Portland, Oregon</td>
<td>I</td>
<td>2123</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>S.W. Barbur Boulevard</td>
<td>Portland, Oregon</td>
<td>II</td>
<td>2937</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>S.E. Powell Boulevard</td>
<td>Portland, Oregon</td>
<td>II</td>
<td>1405</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>E. Speedway Boulevard</td>
<td>Tucson, Arizona</td>
<td>III</td>
<td>950</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Pratt Street</td>
<td>Baltimore, Maryland</td>
<td>IV</td>
<td>410</td>
<td>35</td>
</tr>
</tbody>
</table>
Of the six sites listed in Table G-1, four were used to verify the proposed methodology and engine. The four sites selected are:

- Aviation Parkway
- S.W. Barbur Boulevard
- S.E. Powell Boulevard
- Pratt Street

The first three sites listed have coordinated-actuated control. Pratt Street has coordinated-pretimed operation. These four sites were selected to offer a range in speed limit, access point density, one-way vs. two-way operation, median type, control type, and segment length. It was rationalized that additional sites would not add to the insights obtained from the evaluation of these four sites. Table G-2 summarizes the traffic characteristics and signalization at each of these segments.

### TABLE G-2  Segment traffic and signalization characteristics

<table>
<thead>
<tr>
<th>Street Segment</th>
<th>Travel Direction</th>
<th>Average Volume(^1), veh/h</th>
<th>Base Free-Flow Speed(^2), mph</th>
<th>Average Volume(^1), veh/h</th>
<th>Base Free-Flow Speed(^2), mph</th>
<th>Signal Timing(^1)</th>
<th>Left-Turn Phasing(^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Cycle Length, s</td>
<td>Major Street Split, s</td>
<td>Major Street</td>
<td>Minor Street</td>
<td></td>
</tr>
<tr>
<td>Aviation Parkway</td>
<td>NB</td>
<td>1200</td>
<td>47</td>
<td>80</td>
<td>54</td>
<td>Protected</td>
<td>Protected</td>
</tr>
<tr>
<td></td>
<td>SB</td>
<td>1300</td>
<td>47</td>
<td></td>
<td>54</td>
<td>Permissive</td>
<td>Permissive</td>
</tr>
<tr>
<td>S.W. Barbur Boulevard</td>
<td>NB</td>
<td>530</td>
<td>40</td>
<td>100</td>
<td>52</td>
<td>Protected</td>
<td>Split</td>
</tr>
<tr>
<td></td>
<td>SB</td>
<td>1700</td>
<td>40</td>
<td></td>
<td>54</td>
<td>Permitted</td>
<td>Prot.-Perm.</td>
</tr>
<tr>
<td>S.E. Powell Boulevard</td>
<td>EB</td>
<td>1130</td>
<td>38</td>
<td>120</td>
<td>93</td>
<td>Protected</td>
<td>Permissive</td>
</tr>
<tr>
<td></td>
<td>WB</td>
<td>1147</td>
<td>38</td>
<td></td>
<td>99</td>
<td>Permissive</td>
<td>Permissive</td>
</tr>
<tr>
<td>Pratt Street</td>
<td>EB</td>
<td>1100</td>
<td>29</td>
<td>110</td>
<td>65</td>
<td>Split(^3)</td>
<td>Split(^3)</td>
</tr>
<tr>
<td></td>
<td>WB</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>not applicable - one-way traffic</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1 - Data listed apply to the downstream intersection.
2 - Base free-flow speed is computed using the proposed methodology.
3 - Given one-way operation, all movements on an approach receive green at the same time. This operation is modeled as split phasing; however, it is recognized that the opposing approach has neither traffic nor a phase.

### Process

The evaluation was based on a comparison of performance measures obtained from the proposed methodology with those obtained from a simulation model. CORSIM (version 5.1) was determined to be the most appropriate simulation model for this purpose.

The evaluation process consisted of several steps. Initially, the base free-flow speed was computed using the proposed methodology. This speed was then used as an estimate of the “desired free-flow speed” input required for CORSIM.
As a second step, the field-measured traffic counts and signal timing data were used to create a CORSIM input data file for each study site. Each data file was simulated for one hour to achieve stable estimates of segment performance. The performance measures from each simulation run were then extracted from the simulation model output for subsequent comparison. This step was repeated nine additional times. Each time the signal offset between the intersection pair was increased by one-tenth of the cycle length. This approach was undertaken to facilitate a comparison of performance for a range of arrival types.

As a third step, the field data were used to create an input file for the engine. This file was then submitted to the engine and the output extracted for comparison with the CORSIM measures. The evaluation was repeated for the same offsets used for the CORSIM analysis.

Three performance measures were used to assess the level of agreement between the simulated and predicted segment operation. These three measures are:

- segment through movement control delay,
- segment through movement stop rate, and
- travel speed for through vehicles.

Delay and stop rate were obtained only for the through movement exiting the segment at the boundary intersection.

Control delay and stop percentage are available from the CORSIM output file. Stop percentage is defined as the “the ratio of the number of vehicles that have stopped at least once on a link to the total link trips” (1). The definition of a “stopped” vehicle is not specifically provided in the CORSIM documentation. However, in the discussion of “stopped time per vehicle,” the CORSIM manual implies that a stopped vehicle is one that has slowed to a speed less than 3 ft/s. Hence, the CORSIM stop percentage is comparable to the stop rate computed by the engine, after the former is converted to a stop rate (by dividing it by 100).

The CORSIM stop percentage definition appears to imply that additional stops by the same vehicle due to cycle failure are not considered. If this interpretation is factual, then there is a difference between the CORSIM and engine stop estimates because the latter does count additional stops due to cycle failure. However, any bias that may exist is very small because the four study sites do not have movements that operate near their capacity. Thus, stops due to cycle failure are negligible for this evaluation.

The through movement travel speed estimate provided by CORSIM and by the engine relates to those vehicles that exit the segment as a through movement at the boundary intersection, regardless of how they entered the segment. Thus, the estimates from the two sources are directly comparable.
EVALUATION RESULTS

This part of the appendix describes the results of the evaluation process. The discussion is separated into three sections. Initially, the control delay estimates are compared. Then, the stop percentages are compared. Next, the travel speeds are compared. Finally, a comprehensive evaluation is undertaken where all sites are combined in the examination of a performance measure.

Control Delay

The control delay comparison is shown in Figures G-1, G-2, G-3, and G-4 for Aviation Parkway, S.W. Barbur Boulevard, S.E. Powell Boulevard, and Pratt Street, respectively. Each data point shown represents the average value obtained from CORSIM or the engine for one specific offset. Of each pair of figures, the one on the left side indicates the relationship between delay and offset for the peak travel direction. The figure on the right provides a direct comparison between the CORSIM and engine predictions. It includes the data points for both the peak and the off-peak direction.

The thin line shown in the figures on the right represents an “x = y” line. A data point that falls on this line indicates the engine value equals the simulation value. In general, the data shown in all figures tend to cluster around the “x = y” line and suggest that the engine prediction is in good agreement with that obtained from CORSIM.

The figures on the left side compare the platoon dispersion model used in the methodology with that used in CORSIM. The trends in each figure indicate that the platoon dispersion model in the methodology has a sensitivity to offset that is very similar to that from CORSIM. Deviations were investigated and found to be explained by differences in the amount of dispersion produced by the two models. Specifically, the model used in the methodology tends to predict the arrival of platoons at the downstream intersection slightly sooner than does CORSIM. The deviations are most distinct for Pratt Street where the short spacing and low speed amplified the modeling differences.

Stop Rate

The stop rate comparison is shown in Figures G-5, G-6, G-7, and G-8. Each data point shown represents the average value obtained from CORSIM or the engine for one specific offset. Of each pair of figures, the one on the left side indicates the relationship between stop rate and offset for the peak travel direction. The figure on the right provides a direct comparison between the CORSIM and engine predictions. It includes the data points for both the peak and the off-peak direction.

The trends in Figures G-5, G-6, G-7 and G-8 are similar to those found for delay. The fit is generally good for the range of input offsets and observed stop rates. The sensitivity to offset is good and further confirms that the engine’s platoon dispersion model is reasonable. Deviations found in some of the figures on the left are caused by the platoon model differences noted in the previous section.
Figure G-1. Control delay comparison for Aviation Parkway.

Figure G-2. Control delay comparison for S.W. Barbur Boulevard.

Figure G-3. Control delay comparison for S.E. Powell Boulevard.

Figure G-4. Control delay comparison for Pratt Street.
Figure G-5. Stop rate comparison for Aviation Parkway.

Figure G-6. Stop rate comparison for S.W. Barbur Boulevard.

Figure G-7. Stop rate comparison for S.E. Powell Boulevard.

Figure G-8. Stop rate comparison for Pratt Street.

G-6
Through Travel Speed

The travel speed comparison is shown in Figures G-9, G-10, G-11, and G-12. Each data point shown represents the average value obtained from CORSIM or the engine for one specific offset. Of each pair of figures for a given site, the one on the left side indicates the relationship between travel speed and offset for the peak travel direction. The figure on the right provides a direct comparison between the CORSIM and engine predictions. It includes the data points for both the peak and the off-peak direction.

The trends in Figures G-9, G-10, G-11 and G-12 are similar to those found for delay and stop rate. The fit is generally good for the range of input offsets and observed speeds. The sensitivity to offset is good and further confirms that the platoon dispersion model is reasonable. Deviations found in some of the figures on the left are caused by the differences noted in the section titled Control Delay. It is noted that the engine’s predicted speed for the two shorter segments appears to be a couple miles per hour slower than that predicted by CORSIM.

Combined Evaluation

The delay, stop rate, and travel speed for all sites combined are shown in Figures G-13, G-14, and G-15, respectively. Also shown is the root mean square error for each of the regression analyses. This variable provides an indication of the standard deviation of the prediction. It is based on one hour of simulation for each data point. Smaller values are likely for longer simulation runs. Figure G-13 indicates that the proposed methodology is inclined to yield delays that are 1 to 2 s larger than those predicted by CORSIM. This difference is likely due to the different platoon dispersion models being used, especially in terms of when each predicts the arrival of the platoon.

The stop rate predicted by the methodology is shown in Figure G-14. The quality of fit is similar to that for delay. The methodology is able to explain 88 percent of the variability in the CORSIM predicted stop rate. The standard deviation of the estimate from the methodology is 0.07 stops/veh.

Figure G-15 indicates that the proposed methodology is a reasonably good predictor of the CORSIM travel speed, especially for longer segments with higher speed. It has a tendency to estimate slightly slower speeds on shorter segments with lower speeds. The standard deviation of the estimate is 1.5 mph. The methodology explains about 97 percent of the variability in the CORSIM predicted speeds.
Figure G-9. Travel speed comparison for Aviation Parkway.

Figure G-10. Travel speed comparison for S.W. Barbur Boulevard.

Figure G-11. Travel speed comparison for S.E. Powell Boulevard.

Figure G-12. Travel speed comparison for Pratt Street.
Figure G-13. Control Delay - All Sites. Figure G-14. Stop Rate - All Sites.

Figure G-15. CORSIM Travel Speed - All Sites. Figure G-16. HCM Travel Speed - All Sites.

Figure G-16 compares the travel speed estimated using the methodology in Chapter 15 of the *Highway Capacity Manual (HCM)* with the travel speed predicted by the proposed methodology. The delay model is common to both the *HCM* and the proposed methodology so the comparison is based on the running time predicted by the two methodologies (i.e., the delay used in the travel speed estimate is the same for both methodologies). The trend in Figure G-16 indicates that the proposed methodology is a good predictor of the *HCM* predicted travel speed. In contrast to the trend in Figure G-15, the two methodologies are in agreement on travel speed for shorter segments with slower speeds. Also in contrast to the trend in Figure G-15 is the tendency of the proposed methodology to estimate slightly higher speeds on longer segments (i.e., higher by about 6 percent). It appears that CORSIM and the *HCM* have more disagreement with each other about the true travel speed than with the proposed methodology. The speed predicted by the proposed methodology tends to have a value that is between that obtained from CORSIM and the *HCM*.

**CONCLUSIONS**

The proposed methodology was evaluated by comparing its prediction of delay, stop rate, and travel speed with that obtained from CORSIM. The comparison was based on four segments that collectively offered a range of speed limit, access point density, one-way vs. two-way operation,
median type, control type, and segment length. The findings from the evaluation indicate that the proposed methodology provides a reasonable sensitivity to offset, platoon dispersion, and segment length in its predicted performance measures. The travel speed predicted by the methodology is in good agreement with that obtained from CORSIM and \textit{HCM} Chapter 15.

\textbf{REFERENCES}

APPENDIX H

Proposed Text for the Highway Capacity Manual
CHAPTER 10

URBAN STREET CONCEPTS (Only selected sections)

CONTENTS

I. INTRODUCTION ........................................... 10-1
II. URBAN STREETS ........................................... 10-1
   Urban Street System .................................... 10-2
   Points and Segments ..................................... 10-2
   Segment Length Considerations ....................... 10-3
   Urban Street Facility ................................... 10-3
   Facility Length Considerations ....................... 10-3
   Flow Characteristics .................................... 10-4
   Running Speed ........................................... 10-4
   Free-Flow Speed ........................................ 10-4
   Control Delay ........................................... 10-5
   Stop Rate ............................................... 10-5
   Travel Speed ............................................ 10-6
   Time-Space Trajectory ................................ 10-6
   Levels of Service ...................................... 10-7
   Urban Street Class ..................................... 10-7
   Required Input Data and Estimated Values .......... 10-10
III. SIGNALIZED INTERSECTIONS ......................... 10-11
   Traffic Signal Characteristics ....................... 10-11
   Types of Traffic Signal Control ..................... 10-11
   Intersection Traffic Movements ...................... 10-12
   Signal Phase Sequence ................................ 10-13
   Operational Modes ..................................... 10-14
   Left-Turn Phase Sequence ............................ 10-14
I. INTRODUCTION

Concepts for describing the performance provided to automobile traffic by an urban street are introduced in this chapter. The term “urban streets” refers to arterial, collector, and local streets within areas having a population of about 5,000 persons or more.

Methodologies in Chapter 15 (Urban Streets), Chapter 16 (Signalized Intersections), and Chapter 17 (Unsignalized Intersections) are used in conjunction with this chapter to evaluate the performance provided by an urban street and the intersections on these streets. Intersection performance is evaluated in terms of the delay incurred by intersecting traffic movements. Street performance is evaluated in terms of the speed of, and stops incurred by, vehicles traveling along the street.

II. URBAN STREETS

The urban street system consists of streets that are functionally classified as arterial, collector, and local. The primary purposes of an arterial street are to promote mobility by serving longer through trips and to support the hierarchy of traffic movement by providing connections to other arterial or collector streets (often at signalized intersections). Provision of access to adjacent land uses is often disruptive to arterial street traffic flow and counter to its purpose.

The primary purpose of a collector street is to provide both land access and mobility within residential, commercial, and industrial areas. It also provides a connection to arterial, collector, and local streets using a mix of signalized, stop-controlled, and uncontrolled intersections. Its access function is more important than that of an arterial street.

The primary purpose of a local street is to provide access to adjacent property. It also provides a connection to collector and local streets using a mix of stop-controlled and uncontrolled intersections.

Downtown streets often resemble arterial streets because of frequent signalized intersections and high through volumes. In contrast, they often resemble collector streets because they provide access to local businesses. Turning movements at downtown intersections are often greater than 20 percent of total traffic volume because downtown flow typically involves a substantial amount of circulatory traffic. Pedestrian conflicts and lane obstructions created by stopping or standing taxicabs, buses, trucks, and parking vehicles are typical of downtown streets. Downtown street function may change with the time of day. For this reason, some strategically located downtown streets are converted to arterial-type operation during peak traffic hours.

Multilane suburban and rural highways differ from urban streets in the following ways: roadside development is not as intense, density of access points is lower, and signalized intersections are more than 2 mi apart. These conditions result in fewer traffic conflicts, smoother flow, and nearly random (i.e., non-platooned) arrival patterns.

The performance of an urban street can be described in terms of the mobility and accessibility it provides to travelers. The degree of mobility provided is typically quantified in terms of operational measures of effectiveness, such as travel speed or travel time rate. The degree of accessibility can be quantified in terms of access point density, trip circuity, and network connectivity. Of these two descriptors, the methods described in this manual are focused on the evaluation of mobility. However, they may be helpful in the evaluation of some measures of accessibility.
URBAN STREET SYSTEM

The roadway system is often considered to consist of individual system “elements” that are physically adjacent and operate as a single entity for the purpose of serving travelers and commercial goods carriers. These elements are categorized as “points” or “segments.” A point represents the boundary between segments and is usually represented by an intersection or ramp terminal. A segment represents a length of roadway between two points.

In this manual, the elements of the roadway system form the basic analysis units for which performance can be quantified. A methodology is described for evaluating each analysis unit. Each methodology produces one or more performance measures. These measures provide insight regarding the performance of an individual element. They can also be combined with those of other elements and travel modes to yield indicators of facility or corridor performance.

Points and Segments

The roadway system elements are more specifically defined when applied to the urban street system. This specificity is in recognition of the important influence of intersections on urban street operations. The interaction between the intersection point and street segment is sufficiently complex that they must be evaluated together to provide an accurate indication of urban street performance. In recognition of these interactions, a “street segment” is defined herein to include both the segment and the point at each end of this segment.

If the subject street segment is within a coordinated signal system, then the following rules apply when identifying the segment boundaries:

1. A signalized intersection (or ramp terminal) is always used to define a segment boundary.
2. Only intersections (or ramp terminals) at which the segment through movement is uncontrolled (e.g., a two-way stop controlled intersection) can exist along the segment between the boundaries.

If the subject street segment is not within a coordinated signal system, then the following rules apply when identifying the segment boundaries:

1. An intersection (or ramp terminal) having a type of control that can impose on the segment through movement a legal requirement to stop or yield must always be used to define a segment boundary.
2. An intersection (or ramp terminal) at which the segment through movement is uncontrolled (e.g., a two-way stop controlled intersection) may be used to define a segment boundary, but it is typically not done.

Hereafter, all references to “segment” in Chapters 10 and 15 will imply use of the aforementioned definition of “street segment.”

Previous editions of this manual have defined a segment as representing one direction of travel along a length of roadway between two points. Thus, the length of roadway consisted of two segments when traffic flowed in two directions. This definition was motivated by the mechanics of the computation process associated with previous editions of this manual. Unfortunately, it does not adequately recognize the interactions present when extending the analysis to urban street systems. The platoon formed by the signal at one end of the segment influences the operation of the signal at the other end of the segment. It also influences two-way stop controlled intersections located along the segment.

To accurately evaluate the aforementioned interactions, it is important to evaluate both travel directions on the segment together. For this reason, the segment definition is broadened to include both travel directions.
Segment Length Considerations

When a segment has a “short” length, then the interaction between traffic movements and traffic control devices at the two boundary intersections is sufficiently complex that a separate analysis of each element will not provide an accurate indication of urban street performance. This complication may occur regardless of the type of control present at the two boundary intersections; however, it is particularly complicated when the two intersections are signalized. The methodology described in Chapter 15 is not appropriate for the analysis of short segments. However, the methodology described in Chapter 26 is appropriate for the analysis of short segments at signalized interchanges.

In the context of the previous paragraph, it is difficult to define specific conditions under which a segment is “short.” However, two general rules apply in making this determination. First, a segment is considered to be short if the queue frequently extends back from one intersection into the other intersection (i.e., spills back). Second, a segment is considered to be short if the through signal phase duration at the downstream intersection is longer than that needed to serve the combined number of vehicles that store on the segment plus any that can enter it from the upstream signalized intersection while the downstream phase is green. This situation results in “demand starvation.” It leads to the inefficient use of the downstream through phase and the retention of unserved vehicles on the approaches to the upstream intersection. In general, segments that are bounded by signalized intersections and are shorter than 400 ft may experience one or both of these conditions.

Platoons formed at a signalized intersection are typically dispersed by the time they reach a point about 1 mi downstream of the intersection. This distance can vary, depending on the amount of access point activity along the street and the speed of the traffic stream. Regardless, the influence of platoons on urban street operation is likely to be negligible when segment length exceeds 2 mi. Therefore, if a segment exceeds 2 mi in length and its boundary points are signalized, then the analyst should determine if it is more appropriate to evaluate the segment as an uninterrupted flow highway segment with isolated intersections.

Urban Street Facility

An urban street facility is a contiguous length of roadway composed of two or more connected segments with similar cross section, traffic volume, speed, adjacent land use, and percentage of turning vehicles. At least one intersection (or ramp terminal) along the facility must have a type of control that can impose on the segment through movement a legal requirement to stop or yield. Significant changes in one or more of the facility characteristics may indicate the end of one facility and the start of a second facility. Facility lengths can range from 0.75 to 2.0 mi (typically 1 mi) in urbanized downtown areas and from 1.5 to 5.0 mi (typically 3 mi) in other areas.

Facility Length Considerations

Urban arterial streets and major collector streets are designed to accommodate longer trips than minor collector streets or local streets. They also serve cross-town trips and support the hierarchy of movement by connecting to streets of lower functional class. To serve these purposes adequately, arterial streets and major collector streets should have a length of 1 mi or more in downtown areas and 2 mi or more in other areas. When a street meets or exceeds this length, average travel speed is more meaningful to drivers traveling along the facility and is a useful indicator of facility performance.

If a facility assessment is desired, the analyst will need to evaluate all of the segments that comprise the facility and aggregate the travel time for each segment to obtain a facility travel speed estimate. If the street has two-way traffic flow, then the travel speed for each direction of travel should be separately computed.
The methodology described in Chapter 15 is not limited to facility evaluation. It can also be used to evaluate the performance of an individual segment. It is up to the analyst to determine the scope of the analysis (i.e., one segment, two segments, or all segments on the facility) based on analysis objectives and agency directives.

In some situations, the analyst may be interested in the performance of only a subset of the segments that comprise the facility. For example, the analyst may be interested in the impact of a proposed commercial development on the adjacent street segment. Thus, the impact of the development on traffic progression at the intersections that bound this segment may be of interest. The average travel speed along the segment can also be computed but it should be recognized that this speed may not be a reliable indicator of traveler perception of overall facility performance.

FLOW CHARACTERISTICS

Urban street performance is influenced by three main factors: street environment, traffic characteristics, and presence of traffic control devices. The street environment is described by the geometric characteristics of the roadway, the types of roadside traffic activity, and the adjacent land uses. Specific elements of the street environment include the number and width of traffic lanes, type of median, access-point density, spacing between signalized intersections, existence of on-street parking, level of pedestrian activity, and speed limit.

The character of the traffic stream is described by its density, the proportion of trucks and buses, and turning movement volume. Density relates to the proximity of adjacent vehicles. Drivers tend to adopt slower speeds as the density of the traffic stream increases. Trucks and buses do not accelerate and are more difficult to maneuver on urban streets. As a result, intersection delay tends to increase as the proportion of trucks and buses increase. Turn movements at an access point or signalized intersection tend to require more service time than through vehicles. As a result, the performance of all traffic movements tends to degrade as the proportion of turns increases.

Traffic control devices (including signals and signs) along the street can cause some, or all, vehicles to slow or stop. The delay caused by these devices can be significant and can reduce overall travel speed.

The remainder of this section describes various traffic flow characteristics that are directly related to urban street performance. These characteristics include: running speed, free-flow speed, control delay, stop rate, and travel speed. The influence of the aforementioned factors on speed and delay is also described.

Running Speed

The running speed along a segment is computed as segment length divided by average running time. Average running time equals the time to traverse the segment, less any control delay incurred as a result of compliance with a traffic control device. The character of the traffic stream (i.e., density, proportion trucks, turn volume) and the street environment influence driver speed choice and, thereby, affect running speed.

Running time can be directly measured on segments with signalized boundaries, provided that the signals do not slow or stop the observed traffic stream. Running time cannot be directly measured on a segment with stop or yield-controlled intersections. However, it can be estimated by measuring the running time along a portion of the segment that is not influenced by the boundary intersections and then extrapolating the partial segment running time into an estimate of the total segment running time.

Free-Flow Speed

Free-flow speed represents the average speed of through automobile drivers when traveling along a street under low-volume conditions and when not delayed by any
control device or other vehicle. Free-flow speed reflects the effect of the street environment on driver speed choice.

Running time under free-flow conditions can be directly measured on segments with signalized boundaries provided that: (1) the signals do not slow or stop the observed automobile traffic stream and (2) traffic volumes are low (i.e., less than 200 veh/h/ln). Running time under free-flow conditions cannot be directly measured on a segment with stop or yield-controlled intersections. However, it can be estimated by using the procedure described in the previous section for running speed.

Related to the concept of free-flow speed is “base free-flow speed.” The base free-flow speed is defined to be the free-flow speed on a long street segment. Research indicates that free-flow speed is influenced by segment length. Specifically, free-flow speed is observed to be lower on shorter segments when platoons are unable to disperse and platoon speed is limited to that of the vehicles leading the platoon. This effect is independent of volume, provided that there are enough vehicles to form a platoon of two or more vehicles during each cycle.

Seldom can a driver travel at the free-flow speed. Often, the presence of other vehicles restricts vehicle speed. Drivers may have to slow to adopt the speed of a slower vehicle ahead because vehicles in adjacent lanes preclude the possibility of passing. Also, drivers may slow as they converge on a platoon of vehicles that are accelerating from a signal. For these reasons, running speed tends to be slower than free-flow speed during moderate to high-volume conditions.

**Control Delay**

Control delay can be incurred at any intersection (or ramp terminal) having a type of control that can impose a legal requirement to stop or yield. It represents the additional travel time caused by compliance with a traffic control device.

Of particular interest when evaluating urban street performance is the control delay incurred by the through movement that exits a segment at a boundary intersection (i.e., the segment through movement). For signalized intersections, the extent of control delay incurred by the segment through movement is dependent on the signal timing at the downstream intersection and the quality of traffic progression, as achieved through signal coordination. For unsignalized intersections (including roundabout and all-way stop controlled intersections), the extent of control delay incurred by the segment through movement is dependent solely on the type of control and conflicting traffic volumes present at the downstream intersection.

**Stop Rate**

Traffic control devices separate vehicles on conflicting paths by requiring one vehicle to stop or yield to the other vehicle. The stop causes delay to be incurred and it also has a cost associated with it in terms of fuel consumption and wear on the vehicle. For this reason, information about stops incurred is useful for performance evaluation and the calculation of road user costs. This measure is typically expressed in terms of “stop rate,” which represents the count of stops divided by the number of vehicles served. Stop rate has units of “stops per vehicle.”

Stops are generally expected by motorists when arriving at an intersection as a minor movement (e.g., a turn movement or a through movement on the minor street). However, through drivers do not expect to stop when traveling along a major street. Their expectation is that the signals will be coordinated to some degree such that they can arrive at each signal in succession while it is displaying a green indication for the through movement. For this reason, stop rate is a useful performance measure for evaluating coordinated signal systems.
Travel Speed

The travel speed along an urban street is computed as the length of the segment divided by the average travel time. The travel time represents the sum of the average running time and the segment through movement control delay. Travel speed reflects the effect of adjacent vehicles on driver speed choice, the street environment, and traffic control devices on speed.

Travel time can be directly measured on segments by using the test-car method to obtain a representative sample of segment travel times. It can also be measured by tracking individual vehicles along the segment. A method for measuring travel time in the field is described in Appendix B of Chapter 15.

Time-Space Trajectory

Exhibit 10-1 shows simplified time-space trajectories of representative vehicles along one lane of an urban street segment. The slope of each line at any instant in time reflects the corresponding vehicle speed at that time. Steeper slopes represent higher speeds; horizontal lines represent stopped vehicles.

In Exhibit 10-1, the trajectories for vehicles 1 and 2 indicate that they turned onto the street from the mid-segment access point. The other vehicles shown were discharged from the upstream signal. Vehicles 1, 2, and 3 arrived at the downstream signal during the red interval and had to stop. Vehicle 4 could have arrived at the stop line at the start of the green indication, but had to stop because it was blocked by Vehicle 3, which was not yet in motion. Vehicles 5, 6, and 7 did not stop, but had to reduce their speed because they were affected by the stoppages caused by the signal. Vehicle 8 was slowed by Vehicle 7. The speeds of Vehicles 9 and 10 were not affected by the presence of other vehicles or the downstream traffic control.
LEVELS OF SERVICE

The average travel speed for through vehicles on an urban street is the measure used as the determinant of the operating level of service (LOS). This speed is computed for a segment or an entire urban street facility. It reflects the factors that influence running time along a segment and the delay incurred at the intersections. The focus is on the through driver given that the objective is to evaluate the mobility provided for longer trips. The following paragraphs characterize level of service for an urban street.

LOS A describes primarily free-flow operation. Vehicles are completely unimpeded in their ability to maneuver within the traffic stream. Control delay at signalized intersections is minimal. The average travel speed is 85 percent or more of the base free-flow speed.

LOS B describes reasonably unimpeded operation. The ability to maneuver within the traffic stream is only slightly restricted and control delays at signalized intersections are not significant. The average travel speed is 67 to 84 percent of the base free-flow speed.

LOS C describes stable operation. The ability to maneuver and change lanes at mid-segment locations may be more restricted than at LOS B. Longer queues, adverse signal coordination, or both may contribute to lower average travel speeds. The average travel speed is 50 to 66 percent of the base free-flow speed.

LOS D indicates a less stable condition in which small increases in flow may cause substantial increases in delay and decreases in travel speed. This operation may be due to adverse signal progression, inappropriate signal timing, high volumes, or some combination of these factors. The average travel speed is 40 to 49 percent of the base free-flow speed.

LOS E is characterized by unstable operation and significant delay. Such operations are caused by some combination of adverse progression, high signal density, high volumes, extensive delays at critical intersections, and inappropriate signal timing. The average travel speed is 30 to 39 percent of the base free-flow speed.

LOS F is characterized by flow at extremely low speed. Congestion is likely occurring at critical signalized intersections and is associated with high delays, frequent cycle failures, and extensive queuing. The average travel speed is less than 30 percent of the base free-flow speed.

URBAN STREET CLASS

An urban street classification system is described in this section. It is based on the function and design of the urban street. As noted in a previous section, function and design are indications of the extent to which a street serves the mobility and access needs of travelers. A street’s functional category reflects consideration of the relative importance of mobility and access to travelers on the street. A street’s design category describes the presence or frequency of physical features and traffic controls that directly influence mobility and access. Specifically, the design category depends on the posted speed limit, signal density, driveway/access-point density, and other design features.

There are four urban street classes defined in this manual. They are designated by a number between 1 and 4, using Roman numerals (i.e., I, II, III, and IV). Exhibits 10-2 and 10-3 are useful for determining an urban street’s class. When applied to a specific street, Exhibit 10-2 is consulted first to determine the appropriate functional and design categories. Then, Exhibit 10-3 is used with the design and functional categories to determine which of the four classes best describes the subject street.

The functional category consists of two designations: arterial and collector. An arterial street serves through movements between important centers of activity in a
Highway Capacity Manual 2010

metropolitan area and a substantial portion of trips entering and leaving the area. It also
connects freeways with major traffic generators. In smaller cities (population under
50,000), its importance is derived from the service provided to traffic passing through
the urban area. Access to abutting land is subordinate to the function of moving through
traffic.

A collector street connects and augments the arterial street system. It provides more
of a balance in the mobility and access functions than does the arterial street. It serves
trips of moderate length to geographical areas smaller than those served by the arterial.

The urban street is further classified by its design category. This category consists
of four designations: high-speed, suburban, intermediate, and urban. As shown in
Exhibit 10-2, this classification is based on consideration of segment cross section, signal
spacing, speed, and roadside activity. Illustrations 10-1 through 10-4 show typical
examples of the four urban street design categories.

<table>
<thead>
<tr>
<th>EXHIBIT 10-2. Functional and Design Categories</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Criterion</strong></td>
</tr>
<tr>
<td>Mobility function</td>
</tr>
<tr>
<td>Access function</td>
</tr>
<tr>
<td>Points connected</td>
</tr>
<tr>
<td>Predominant trips served</td>
</tr>
</tbody>
</table>

| **Criterion** | **Design Category** | **High-speed** | **Suburban** | **Intermediate** | **Urban** |
| Access density | Very low | Low | Moderate | High |
| Cross section | Divided or undivided | Divided or undivided | Divided or undivided | Undivided |
| Travel directions | Two-way | Two-way | One-way or two-way | One-way or two-way |
| On-street parking | None | None | Some | Significant |
| Separate left-turn lanes | Yes | Yes | Usually | Some |
| Signal spacing | 0.5 to 2.0 mi | 0.2 to 1.0 mi | 530 to 1300 ft | 440 to 880 ft |
| Signal density | 0.5 to 2 sig/mi | 1 to 5 sig/mi | 4 to 10 sig/mi | 6 to 12 sig/mi |
| Speed limit | 45 to 55 mph | 40 to 45 mph | 30 to 40 mph | 25 to 35 mph |
| Pedestrian activity | Very little | Little | Some | Usually |
| Development density | Low | Low to medium | Medium to high | High |

<table>
<thead>
<tr>
<th>EXHIBIT 10-3. Urban Street Class</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design Category</strong></td>
</tr>
<tr>
<td>High-Speed</td>
</tr>
<tr>
<td>Suburban</td>
</tr>
<tr>
<td>Intermediate</td>
</tr>
<tr>
<td>Urban</td>
</tr>
</tbody>
</table>
ILLUSTRATION 10-1. Typical high design

ILLUSTRATION 10-2. Typical suburban design.

ILLUSTRATION 10-3. Typical intermediate design.
ILLUSTRATION 10-4. Typical urban design.

High speed design represents an urban street with a very low access-point density, separate left-turn lanes, and no parking. The street may have a multilane divided cross section or an undivided two-lane cross section. Signals are infrequent and spaced at long distances. Roadside development density is low, and the speed limit is typically 45 to 55 mph. This design category often describes urban streets in suburban settings.

Suburban design represents a street with a low access-point density, separate left-turn lanes, and no parking. The street may have a multilane divided cross section or an undivided two-lane cross section. Signals are spaced for good progressive movement. Roadside development density is low to medium and speed limits are usually 40 to 45 mph.

Intermediate design represents an urban street with a moderate access-point density. It may have a multilane divided cross section, a multilane undivided one-way cross section, or an undivided two-lane cross section. The street may have some separate or continuous left-turn lanes. Parking may be permitted along some portion of its length. It has a higher density of roadside development than the typical suburban design and usually has 4 to 10 signals per mile. Speed limits are typically 30 to 40 mph.

Urban design represents an urban street with a high access-point density. The street frequently is an undivided one-way or two-way facility with two or more lanes. Parking is usually permitted. Generally, there are few separate left-turn lanes, and some pedestrian interference is present. It commonly has 6 to 12 signals per mile. Roadside development is dense with commercial uses. Speed limits range from 25 to 35 mph.

REQUIRED INPUT DATA AND ESTIMATED VALUES

[This section may be deleted or re-written depending on the extent to which the HCQS committee incorporates the recommendations from NCHRP Project 3-82]
III. SIGNALIZED INTERSECTIONS

The operational performance of automobiles traveling along an urban street is influenced by the geometric design of the roadway and the intersections that comprise the street. It is also influenced by the traffic control devices used at the intersections. This influence is most notable when a traffic control signal is used because delay can be significant and vary widely among the intersecting movements. The signal timing plan used to progress traffic along the street can be a dominant factor in the level of service provided to motorists traveling along the street.

Traffic demand along an urban street typically varies in volume and pattern during the course of the day and from month to month. In recognition of these changes and their potential adverse impact on urban street operation, the traffic control signal’s timing plan is often varied over time based on detector inputs, time-of-day programming, or external instructions from a master controller or management center.

At its most fundamental level, a traffic signal essentially allocates time among conflicting traffic movements that seek to use the same physical space on a roadway. The means by which this time is allocated to the conflicting movements will significantly affect the operation of the intersection and the traffic it serves.

TRAFFIC SIGNAL CHARACTERISTICS

This section provides an overview of traffic signal controller characteristics and operation. Its purpose is to supplement the discussion in Chapters 15 and 16 by providing some basic concepts that are fundamental to the understanding of controller operation and the evaluation of intersection operation. The topics addressed include: signal control type, intersection traffic movements, signal phase sequence, left-turn operational modes, and left-turn phase sequence.

Types of Traffic Signal Control

All signal controllers use the concept of a “phase,” where a phase is defined as the combination of green, yellow change, and red clearance intervals used to control one or more intersection traffic movements. In general, a controller will operate as pretimed or actuated. These two types of control are described as follows:

- **Pretimed control** consists of a fixed sequence of phases that are displayed in repetitive order. The duration of each phase is fixed. However, the green interval duration can be changed by time of day or week to accommodate traffic variations. The combination of a fixed phase sequence and duration produces a constant cycle length.

- **Actuated control** consists of a defined phase sequence wherein the presentation of each phase is dependent on whether the associated traffic movement has submitted a call for service through a detector. The green interval duration is determined by the traffic demand information obtained from the detector, subject to preset minimum and maximum limits. The termination of an actuated phase requires a call for service from a conflicting traffic movement. An actuated phase may be skipped if no demand is detected.

The operation of an actuated controller can be described as fully-actuated, semi-actuated, or coordinated-actuated. These control variations are described as follows:

- **Fully-actuated control** implies that all phases are actuated and all intersection traffic movements are detected. The sequence and duration of each phase is determined by traffic demand. Hence, this type of control is not associated with a constant cycle length.

- **Semi-actuated control** uses actuated phases to serve the minor movements at an intersection. Only these minor movements have detection. The phases associated with the major movements are operated as “non-actuated.” The controller is
programmed to dwell with the non-actuated phases displaying green for at least a specified minimum duration. The sequence and duration of each actuated phase is determined by traffic demand. Hence, this type of control is not associated with a constant cycle length.

- Coordinated-actuated control is a variation of semi-actuated operation. It uses the controller’s force-off settings to constrain the non-coordinated phases associated with the minor movements such that the coordinated phases are served at the appropriate time during the signal cycle and progression for the major movements is maintained. This type of control is associated with a constant cycle length.

Signalized intersections that are close to one another along a street are often operated as a coordinated signal system, where specific phases at each intersection are operated on a common time schedule to permit the continuous flow of the associated movements at a planned speed. The signals in a coordinated system typically operate using pretimed or coordinated-actuated control and the coordinated phases typically serve the major-street through movements. Signalized intersections that are not part of a coordinated system are characterized as “isolated” and typically operate using fully-actuated or semi-actuated control.

### Intersection Traffic Movements

Exhibit 10-a illustrates typical vehicle and pedestrian traffic movements at a four-leg intersection. Three vehicular traffic movements and one pedestrian traffic movement are shown for each intersection approach. Each movement is assigned a unique number, or a number and letter combination. The letter “R” denotes a right-turn movement. The letter “P” denotes a pedestrian movement. The cardinal direction shown in the exhibit is for convenience of this discussion. The assignment of numbers to the major and minor street movements as shown is typically maintained in practice, regardless of whether the major street is oriented in a north-south or east-west direction. The movement numbers for the left-turn and through movements follow the NEMA phase numbering convention.

Intersection traffic movements are assigned the right-of-way by the signal controller in real time. Each movement is assigned to one or more signal phases. The engineer's assignment of movements to phases will vary in practice, depending on the desired phase sequence and the movements that are present at the intersection.
Signal Phase Sequence

Early controllers implemented signal phasing using a single-ring structure where all non-conflicting traffic movements were assigned to a common phase and its duration was dictated by the one movement needing the most time. Modern actuated controllers implement signal phasing using a dual-ring structure that allows for the concurrent presentation of a green indication to two phases. Each phase serves one or more movements that do not conflict with each other or those of the concurrent phase.

Of the two ring structures, the dual-ring structure is more efficient because it allows the controller to adapt phase duration and sequence to the needs of the individual movements. The dual-ring structure is typically used with eight phases; however, more phases are available for complex signal phasing. The eight-phase dual-ring structure is shown in Exhibit 10-b. The symbol “Φ” shown in this figure represents the word “phase” and the number following the symbol represents the phase number.

EXHIBIT 10-b. Dual-Ring Structure with Illustrative Movement Assignments.

Shown in Exhibit 10-b are the traffic movements sometimes assigned to each of the eight phases. These assignments are illustrative, but they are not uncommon. Each left-turn movement is shown to be assigned to a phase. During this phase, the left-turn movement is “protected” such that it receives a green arrow indication. Each through, right turn, and pedestrian movement combination is also assigned to a phase. The dashed arrows indicate turn movements that are served in a “permissive” manner such that the turn can be completed only after yielding the right-of-way to conflicting protected movements.

Two rings and two barriers are identified in Exhibit 10-b. A ring consists of two or more sequentially timed conflicting phases. Ring 1 consists of phases 1, 2, 3, and 4. Ring 2 consists of phases 5, 6, 7, and 8. A barrier is used when there are two or more rings. It represents a reference point in the cycle at which one phase in each ring must reach a common point of termination. In Exhibit 10-b, a barrier is shown following phases 2 and 6. A second barrier is shown following phases 4 and 8. Between barriers, only one phase in each ring can be active at a time.

The ring structure dictates the sequence of phase presentation. Some common rules are provided in the following list:

- Phase pairs 1-2, 3-4, 5-6, and 7-8 typically occur in sequence. Thus, phase pair 1 and 2 begins with phase 1 and ends with phase 2. However, within each phase pair, it is possible to provide a lagging left turn phase by reversing the order of the pair.
Thus, the pair 1 and 2 could be set to begin with phase 2 and end with phase 1 if it was desired to have the left-turn phase 1 lag through phase 2.

- Phase pair 1 and 2 can operate concurrently with phase pair 5 and 6. That is, phase 1 (or phase 2) can time with either phase 5 or 6. Phase pair 3 and 4 can operate concurrently with phase pair 7 and 8. Phases, 1, 2, 5, and 6 form a concurrency group. Phases 3, 4, 7, and 8 form a concurrency group.
- For a given concurrency group, the last phase to occur in one phase pair must end at the same time as the last phase to occur in the other pair (i.e., end at the barrier).
- Phases that time between the two barriers are typically assigned to the movements on a common street (i.e., major street or minor street).

**Operational Modes**

There are three operational modes for the turn movements at an intersection. The names used to describe these modes refer to the manner in which the turn movement is served by the controller. The three modes are:

- **Permissive**
- **Protected**
- **Protected-Permissive**

The permissive mode requires turning drivers to yield to conflicting traffic streams before completing the turn. Permissive left-turning drivers yield to oncoming vehicles and permissive right-turning drivers yield to pedestrians. The efficiency of this mode is dependent on the availability of gaps in the conflicting streams. An exclusive turn lane may be provided, but it is not required. The permissive turn movement is typically presented with a circular green indication (although some agencies use other indications, such as a flashing yellow arrow). The right-turn movements in Exhibit 10-b are operating in the permissive mode.

The protected mode allows turning drivers to travel through the intersection while all conflicting movements are required to stop. This mode provides for efficient turn movement service; however, the addition of a turn phase typically results in increased delay to the other movements. An exclusive turn lane is typically provided with this mode. The turn phase is indicated by a green arrow signal indication. Left-turn movements 3 and 7 in Exhibit 10-b are operating in the protected mode.

The protected-permissive mode represents a combination of the permissive and protected modes. Turning drivers have the right-of-way during the exclusive phase. They can also complete the turn “permissively” when the adjacent through movement receives its circular green indication. This mode provides for efficient turn movement service, often without causing a significant increase in the delay to other movements. Left-turn movements 1 and 5 in Exhibit 10-b are operating in the protected-permissive mode.

In general, the operational mode used for one left-turn movement is also used for the opposing left-turn movement. For example, if one left-turn movement is permissive, so is the other left-turn. However, this agreement is not required.

**Left-Turn Phase Sequence**

This section describes the sequence of service provided to left-turn movements, relative to the other intersection movements. The typical options include:

- **No Left-Turn Phase (Permissive-Only)**
- **Leading Left-Turn**
- **Lagging Left-Turn**
- **Split**

The No Left-Turn Phase option is used when the left-turn operates in the permissive mode. A left-turn phase is not provided with this option. An illustrative implementation
of permissive-only phasing for left- and right-turning traffic is shown in Exhibit 10-c for the minor street.

Leading, lagging, or split phasing is used when the left-turn operates in the protected or protected-permissive mode. The terms “leading” and “lagging” indicate the order in which the left-turn phase is presented, relative to the conflicting through movement. Leading left-turn phasing was shown previously in Exhibit 10-b for both left-turn movements on both the major and minor streets. Lagging left-turn phasing is shown in Exhibit 10-c for both left-turn movements on the major street. A mix of leading and lagging phasing (called lead/lag) is shown in Exhibit 10-d for the left-turn movements on the major street.

**EXHIBIT 10-c. Illustrative Lag/Lag and Permissive-Only Phasing.**

When the protected-permissive mode is used with lead-lag or lag-lag phasing, then the “yellow trap” (or left-turn trap) problem may occur for one or both of the left-turn movements. This problem stems from the potential conflict between left-turn vehicles and oncoming vehicles at the end of the adjacent through phase. Of the two through movement phases serving the subject street, the trap is associated with the first through movement phase to terminate and occurs during this phase’s yellow change interval. The
left-turn driver seeking a gap in oncoming traffic during the through phase, first sees the yellow ball indication; then incorrectly assumes that the oncoming traffic also sees a yellow indication; and then turns across the oncoming traffic stream without regard to the availability of a safe gap. The “yellow trap” problem can be alleviated by using one of the following techniques:

- Use the protected mode for both left-turn movements.
- Use a “Dallas Display” for both left-turn signal heads and protected-permissive mode for both left-turn movements.

The second technique avoids the yellow trap by using an overlap in the controller and a five-section left-turn signal head. An overlap is a controller output (to the signal head load switch) that is associated with two or more phases. The left-turn green, yellow, and red ball indications are associated with the opposing and adjacent through movement phases using an overlap. The left-turn signal head uses louvers on the yellow and green ball indications to prevent through movement drivers from viewing the left-turn display. The louvered signal head is referred to as the “Dallas Display.” With this display, both left-turn phases can operate in the protected-permissive mode and the trap is avoided.

Split phasing describes a phase sequence where one phase serves all movements on one approach and a second phase serves all movements on the other approach. Split phasing requires that all approach movements simultaneously receive a green indication. Split phasing is shown in Exhibit 10-d for the minor street, other variations may be used depending on the treatment of the pedestrian movements. The left-turn movement in a split phase typically operates in the protected mode (as shown), provided that there are no conflicting pedestrian movements.
## CHAPTER 15

**URBAN STREETS**

### CONTENTS

I. **INTRODUCTION** .......................................................... 15-1  
   Scope ................................................................. 15-1  
   Limitations .......................................................... 15-1  

II. **METHODOLOGY** .......................................................... 15-1  
    Framework ............................................................ 15-2  
    Input Data Requirements ........................................... 15-4  
    Traffic Characteristics Data ...................................... 15-6  
    Geometric Design Data ............................................. 15-9  
    Left-Turn Phasing Data .......................................... 15-11  
    Controller Settings ............................................... 15-11  
    Other Data .......................................................... 15-14  
    Determining Traffic Demand Adjustments ....................... 15-14  
    Capacity Constraint ............................................... 15-15  
    Volume Balance .................................................... 15-15  
    Origin-Destination Distribution .................................. 15-15  
    Spillback Occurrence .............................................. 15-16  
    Determining Running Time ......................................... 15-16  
    Free-Flow Speed .................................................... 15-17  
    Adjustment for Vehicle Proximity ................................ 15-18  
    Additional Running Time Due to Mid-Segment Delay Sources 15-19  
    Determining Proportion Arriving During Green ............... 15-20  
    Determining Signal Phase Duration .............................. 15-20  
    Coordinated Phase Duration ....................................... 15-21  
    Non-Coordinated Phase Duration .................................. 15-21  
    Actuated Phase Duration .......................................... 15-21  
    Detection Design ................................................... 15-22  
    Determining Through Control Delay ............................... 15-23  
    Delay Equation ..................................................... 15-24  
    Progression Adjustment Factor .................................. 15-25  
    Upstream Filtering Adjustment Factor ........................... 15-25  
    Available Capacity ................................................ 15-25  
    Determining Through Stop Rate .................................. 15-26  
    Stop Rate Equation ................................................ 15-27  
    Second-Term Incremental Factor .................................. 15-28  
    Acceleration-Deceleration Delay ................................ 15-28  
    Determining Travel Speed ......................................... 15-29  
    Determining Spatial Stop Rate ................................... 15-29  
    Determining Level of Service ..................................... 15-30  

III. **APPLICATIONS** .......................................................... 15-30  
    Level of Analysis ................................................... 15-30  
    Segmenting the Urban Street ....................................... 15-31  
    Computational Steps ............................................... 15-31  
    Coordinated Segment - Planning Analysis ....................... 15-32  
    Non-Coordinated Segment Analysis ................................ 15-38  
    Coordinated Segment - Operational Analysis ................... 15-40  

IV. **EXAMPLE PROBLEMS** .................................................. 15-43  
    Example Problem 1 .................................................. 15-44  
    Example Problem 2 .................................................. 15-45
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>V. REFERENCES</td>
<td>15-52</td>
</tr>
<tr>
<td>APPENDIX A. DETAILED COMPUTATIONAL MODULES</td>
<td>15-53</td>
</tr>
<tr>
<td>Overview</td>
<td>15-53</td>
</tr>
<tr>
<td>Module Description</td>
<td>15-53</td>
</tr>
<tr>
<td>Demand Adjustment Module</td>
<td>15-53</td>
</tr>
<tr>
<td>Segment Analysis Module</td>
<td>15-55</td>
</tr>
<tr>
<td>Signalized Intersection Module</td>
<td>15-60</td>
</tr>
<tr>
<td>Delay Due to Turns Module</td>
<td>15-67</td>
</tr>
<tr>
<td>Performance Measures Module</td>
<td>15-71</td>
</tr>
<tr>
<td>APPENDIX B. FIELD STUDY PROCEDURE</td>
<td>15-73</td>
</tr>
<tr>
<td>APPENDIX C. WORKSHEETS</td>
<td>15-75</td>
</tr>
</tbody>
</table>
I. INTRODUCTION

This chapter describes a methodology for evaluating the operational performance of automobiles when traveling along an urban street. The methodology can be used to evaluate a segment or a facility, as defined in Chapter 10. The methodology consists of a series of computations based on analytic models of urban street traffic flow processes. The computations are used to estimate performance in terms of travel speed and stop rate for individual segments. Segment performance can be aggregated to obtain an estimate of facility performance. It is envisioned that this methodology will be used in the planning, design, and operation of urban streets for the purpose of properly sizing the urban street, evaluating alternative street and intersection configurations, and establishing efficient signal timing plans.

SCOPE

The methodology can be used to evaluate the mobility function of an urban street in terms of the average travel speed of the automobile traffic stream. Accessibility to abutting property by way of automobile cannot be directly evaluated using this methodology. However, it can be evaluated to some degree through an analysis of the individual intersections along the street. Regardless, a street’s accessibility should also be considered when evaluating its performance, especially if the street is intended to provide such access. Oftentimes, factors that favor mobility reflect minimal levels of access and vice versa.

The methodology is applicable to the evaluation of “automobile traffic.” In this regard, automobile traffic is implied to include a mix of motorized vehicles (including passenger cars, motorcycles, buses, and trucks) with a distribution that reflects typical urban street traffic streams.

The methodology is based on the analysis of steady state traffic conditions and, as such, is not well suited to the evaluation of congested conditions where the queue from one intersection consistently spills back into another intersection.

LIMITATIONS

The urban streets methodology does not directly account for the following conditions on the street that may occur between intersections:
- on-street parking activity;
- grade;
- capacity constraints between intersections (such as a narrow bridge);
- queuing at one intersection consistently backing up to and interfering with the operation of an upstream intersection or access point;
- stops incurred by segment through vehicles due to left or right turns from the segment into an access point (i.e., unsignalized public street or driveway); and
- cross-street congestion blocking through traffic.

In addition, any limitations associated with the methodologies used to evaluate the intersections that bound the urban street segment are shared with this methodology. These limitations are identified in Chapters 16 and 17.

II. METHODOLOGY

This part of the chapter provides an overview of the methodology for evaluating urban street performance. It was developed to support planning, preliminary design, and
operational levels of analysis. Application of the methodology for an operational analysis is the most computationally intense and requires software to implement. The intensity stems from the need to model all intersecting traffic movements and associated signalization at each boundary intersection.

Key procedures have been extracted from the methodology for use in the planning or preliminary design analysis levels. Other procedures have been eliminated by requesting that the planner or designer provide the associated information as input data. Finally, the focus of the planning and preliminary design analysis is just the segment through traffic movement. These simplifying steps allow the planning and preliminary design analysis to be undertaken as a series of manual computations.

The methodology has been developed to evaluate coordinated signal systems. As such, the focus of the discussion in this chapter is on the use of the methodology to evaluate coordinated signal systems. However, as appropriate, the discussion is extended to describe how key elements of this methodology can be used to evaluate non-coordinated systems.

The discussion in this part of the chapter is intended to introduce the analyst to the calculation process, identify the input data requirements, and discuss the key analytic procedures. The procedures described in this part are presented at a level of detail sufficient for a planning or preliminary design analysis. A detailed discussion of the procedures used for an operational analysis is provided in Appendix A. The computational engine maintained by the Highway Capacity and Quality of Service committee represents the most detailed description of this methodology.

FRAMEWORK

Exhibit 15-1 illustrates the framework of the urban streets evaluation methodology. The framework identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. The methodology is shown as it would be applied to one direction of travel along one urban street segment. It should be repeated for the evaluation of the other direction of travel if the street has two-way traffic flow.

The shapes used in the exhibit indicate the type of activity undertaken at each step of the calculation process. The parallelograms indicate steps in the process where input data are needed. The rectangles with rounded corners indicate steps in the process where a procedure is followed to estimate a key variable used in the evaluation. The diamond shape indicates a decision point.

The framework illustrates the calculation process as applied to two system types: coordinated and non-coordinated. The analysis of coordinated systems recognizes the influence of an upstream signalized intersection on the performance of the street segment. The analysis of non-coordinated systems is based on the assumption that arrivals to each boundary intersection are random. As noted previously, the methodology was developed to evaluate coordinated signal systems (i.e., the portion of the framework shown on the right of Exhibit 15-1). However, key elements of the methodology can be extended to the analysis of non-coordinated systems (as shown in the portion of the framework on the left of Exhibit 15-1).

For each of the two system types, the framework is subdivided into the planning and operational analysis levels. The preliminary design analysis level follows that shown for planning. The planning and preliminary design analyses focus on an evaluation of the segment through movement and do not require detailed input data. In fact, the planning and preliminary design analyses may use default values to further reduce input data needs. In contrast, an operational analysis considers all intersection traffic movements and related signal timing. As such, detailed inputs are needed to describe the flow characteristics, geometry, and signal timing associated with each movement.

The framework is further subdivided into the type of traffic control used at the intersections that bound the segment. This approach recognizes that a segment boundary
can be a signalized intersection, roundabout, or all-way stop-controlled intersection. Although not shown, the boundary intersection could also be an interchange ramp terminal. However, the methodology described in Chapter 26 is more appropriate for the evaluation of segments bounded by signalized interchange ramp terminals.

EXHIBIT 15-1. Urban Street Methodology Framework.

Examination of the framework indicates that the operational analysis of a coordinated-actuated street system consists of the largest number of computational steps. This level of analytic detail is dictated by the need to accurately model the many traffic flow processes that occur along a coordinated urban street as well the operation of the actuated traffic signal controller. For planning and preliminary design analyses, several of these steps are omitted and default values are substituted for the variables that would otherwise be computed for an operational evaluation. These steps are described in subsequent sections. Steps that apply only to an operational analysis are presented at a conceptual level, with reference to Appendix A for more detail. The sequence of
calculations are described at a level of detail suitable for a planning or preliminary design analysis, with reference to Appendix A for details about the calculations appropriate for an operational analysis.

Finally, it should be noted in Exhibit 15-1 that there is reference to various procedures described in Chapters 16, 17, and a forthcoming chapter addressing roundabout intersections. With regard to Chapter 16, the procedure for estimating actuated phase duration at isolated intersections is needed for the analysis of fully-actuated and semi-actuated intersections on non-coordinated street systems. Also, the procedure for estimating control delay in Chapter 16 is needed for the estimation of segment through movement delay. The delay estimation procedure for roundabouts and all-way stop-controlled intersections is needed from their respective chapters for the analysis of non-coordinated systems.

**INPUT DATA REQUIREMENTS**

This section describes the input data needed to evaluate an urban street segment. These data are used by one or more of the procedures that comprise the evaluation methodology. They are listed in Exhibit 15-2 and are identified as “data elements.”

Exhibit 15-2 also indicates the procedure within which each specific input data element is used. This use is indicated by the presence of a letter in the cell associated with given combination of data element and procedure. A blank cell indicates that the data element is not a direct input to the procedure.

The letters shown in a cell are an indication of the system type and the level of analysis for which data are needed. The first letter of any pair of letters indicates the system type. The letter “c” indicates data needed to evaluate a coordinated system. The letter “n” indicates the data needed to evaluate a non-coordinated system.

The second letter of any pair of letters indicates the level of analysis. The letter “o” indicates the data are needed for an operational analysis. This analysis level considers all intersecting traffic movements at the boundary intersections. The letter “p” indicates the input data needed for a planning or preliminary design analysis. This analysis level focuses on the evaluation of the segment through movement at the boundary intersections. The word “all” indicates that the data are needed for all combinations of analysis level and system type.

Most of the controller setting data elements listed in Exhibit 15-2 are used in the phase duration estimation procedure for an operational analysis of coordinated-actuated control. A subset of these data is needed for an operational analysis of non-coordinated control. Only cycle length and green-to-cycle-length ratio are needed for a planning analysis.

If the intersection is part of a non-coordinated signal system, then the procedures in Chapter 16 should be used with the input data identified in the exhibit. Additional input data may be needed, as indicated in Chapter 16. If the intersection operates with fully-actuated control, then the procedure described in Appendix B of Chapter 16 should be used to estimate the average phase duration.

If the intersection is part of a non-coordinated system with roundabout or all-way stop controlled intersections, then the procedures in Chapter 17 should be used in the evaluation. The input data needed for this procedure is identified in Chapter 17.

The data elements listed in Exhibit 15-2 do not include variables that are considered to represent calibration factors (e.g., start-up lost time). Default values are typically used for these factors because they have a small impact on the accuracy of the performance estimates produced by the methodology.

The remainder of this section is devoted to a brief discussion of key data elements listed in Exhibit 15-2. Only some of these data are needed for the evaluation of non-coordinated systems or for a planning analysis of coordinated systems. In contrast, most of the data listed are needed for an operational analysis of coordinated systems.
<table>
<thead>
<tr>
<th>Data Category</th>
<th>Input Data Element</th>
<th>Required Data by Procedure¹</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Demand Adjustment</td>
<td>Running Time</td>
</tr>
<tr>
<td>Traffic Characteristics</td>
<td>Movement volume at boundary intersection</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Movement volume at mid-seg. access point</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Platoon ratio for segment through lane group</td>
<td>no</td>
</tr>
<tr>
<td></td>
<td>Platoon ratio for external signalized lane groups</td>
<td>co, no</td>
</tr>
<tr>
<td></td>
<td>Initial queue</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Analysis period length</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Mid-segment delay</td>
<td></td>
</tr>
<tr>
<td>Geometric Design</td>
<td>Number of through lanes on segment</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Number of lanes at boundary intersection</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Number of lanes at mid-seg. access point</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Turn bay length</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Saturation flow rate</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Segment length</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Intersection width</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Restrictive median length</td>
<td>all</td>
</tr>
<tr>
<td></td>
<td>Non-restrictive median length</td>
<td>all</td>
</tr>
<tr>
<td></td>
<td>Percent segment with curb</td>
<td>all</td>
</tr>
<tr>
<td></td>
<td>Number of access points on segment</td>
<td>all</td>
</tr>
<tr>
<td>Left-Turn Phasing</td>
<td>Operational mode</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Phase sequence</td>
<td></td>
</tr>
<tr>
<td>Controller Settings</td>
<td>Minimum green</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Yellow + red clearance</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Passage time</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cycle length</td>
<td>co</td>
</tr>
<tr>
<td></td>
<td>Phase recall</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ring entry mode</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Simultaneous gap-out</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Phase splits</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Offset</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Offset reference point</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Force mode</td>
<td></td>
</tr>
<tr>
<td></td>
<td>g/C ratio for segment through phase</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dallas phasing</td>
<td></td>
</tr>
<tr>
<td>Other</td>
<td>Stop line detector length</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Speed limit</td>
<td></td>
</tr>
</tbody>
</table>

Note:
Traffic Characteristics Data

This section describes the traffic characteristics data listed in Exhibit 15-2. These data describe the automobile traffic stream during the analysis period.

Movement Volume at Boundary Intersection

The movement volume at a boundary intersection represents the count of vehicles arriving to the boundary intersection during the analysis period for the purpose of completing a specific traffic movement. It is expressed in units of vehicles per hour.

Movement Volume at Mid-Segment Access Point

The movement volume at a mid-segment access point represents the count of vehicles arriving to the access point during the analysis period for the purpose of completing a specific traffic movement. It is expressed in units of vehicles per hour.

For an operational analysis of a coordinated system, a volume is needed for each of the movements on the major street and the side street or driveway at each “active” access point. An access point is considered to be “active” if it has sufficient volume as to have some impact on segment operations. As a rule-of-thumb, an access point is considered “active” if it has an entering volume of 10 veh/h or more during the analysis period. If the segment has many inactive access points, they can be combined into one “equivalent” active access point. The equivalent access point has movement volumes that are equal to the sum of the corresponding movement volumes at the individual inactive access points.

For planning and preliminary analyses (or the operational analysis of a non-coordinated system), it is sufficient to estimate the approach volume at a typical access point. The approach volume represents the sum of the left-turn, through, and right-turn volume on the segment approach to a representative access point. An approach volume is needed for each direction of travel.

Platoon Ratio for Segment Through Lane Group

Platoon ratio is used to describe the quality of signal progression for the corresponding lane group. It represents the ratio of the average flow rate during the green indication to the average flow rate during the cycle. This input is needed when the boundary intersection is signalized. It can be computed using the following equation:

\[
R_p = \frac{\text{Arrival Type}}{3}
\]  

where, 
\[
R_p = \text{platoon ratio.}
\]

Arrival type is used to characterize progression quality. Values of arrival type range from 1 to 6. Exhibit 15-3 provides an indication of the quality of progression associated with each arrival type. Each arrival type and progression quality descriptor is described in more detail in the subsequent paragraphs.

Arrival Type 1 is characterized by a dense platoon of more than 80 percent of the lane group volume arriving at the start of the red interval. This arrival type is often associated with shorter segments. If the two signals are not coordinated, then poor progression will likely exist for both travel directions. If coordination is provided for only one direction of travel, then poor progression may exist for the other direction.

Arrival Type 2 is characterized by a moderately dense platoon arriving in the middle of the red interval or a dispersed platoon containing 40 to 80 percent of the lane group volume arriving throughout the red interval.
EXHIBIT 15-3. Relationship between Platoon Ratio and Progression Quality.

<table>
<thead>
<tr>
<th>Arrival Type</th>
<th>Range of Platoon Ratio</th>
<th>Default Platoon Ratio</th>
<th>Progression Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00 to 0.50</td>
<td>0.333</td>
<td>Very poor</td>
</tr>
<tr>
<td>2</td>
<td>0.51 to 0.85</td>
<td>0.667</td>
<td>Unfavorable</td>
</tr>
<tr>
<td>3</td>
<td>0.86 to 1.15</td>
<td>1.000</td>
<td>Random arrivals</td>
</tr>
<tr>
<td>4</td>
<td>1.16 to 1.50</td>
<td>1.333</td>
<td>Favorable</td>
</tr>
<tr>
<td>5</td>
<td>1.51 to 2.00</td>
<td>1.667</td>
<td>Highly favorable</td>
</tr>
<tr>
<td>6</td>
<td>2.01 or more</td>
<td>2.000</td>
<td>Exceptional</td>
</tr>
</tbody>
</table>

Arrival Type 3 describes one of two conditions. If the signals bounding the segment are coordinated, then this arrival type is characterized by a platoon containing less than 40 percent of the lane group volume arriving partially during the red interval and partly during the green interval. If the signals are not coordinated, then this arrival type is characterized by platoons that arrive at different points in time at the subject intersection over the course of the analysis period such that arrivals are effectively random.

Arrival Type 4 is characterized by a moderately dense platoon arriving in the middle of the green interval or a dispersed platoon containing 40 to 80 percent of the lane group volume arriving throughout the green interval.

Arrival Type 5 is characterized by a dense platoon of more than 80 percent of the lane group volume arriving at the start of the green interval. This arrival type may occur on shorter segments with a low-to-moderate number of side street entries and good signal coordination.

Arrival Type 6 is characterized by a dense platoon of more than 80 percent of the lane group volume arriving during the green interval. This arrival type occurs only on short segments with negligible side street entries and near-ideal signal coordination. It is reserved for exceptional progression quality on routes with near-ideal characteristics.

For planning and preliminary design analyses, platoon ratio is input for the lane group serving segment through traffic. For an operational analysis, the platoon ratio for the segment through lane group is computed and does not need to be input.

Platoon Ratio for External Signalized Lane Groups

A platoon ratio is needed for each lane group on an external (i.e., minor street) approach at each boundary intersection that is signalized. This ratio should describe the quality of progression on the minor street segments that intersect with the subject segment. The discussion provided in the previous section can be used to guide the selection of this ratio.

Initial Queue

The initial queue represents the average number of vehicles present in the lane group at the start of the red indication. This average should be based on the initial queue observed for several cycles prior to the start of the analysis period.

Analysis Period Length

The analysis period represents the interval of time considered in an operational evaluation. The duration of this time period is typically in the range of 0.25 to 1 h. Shorter durations do not provide sufficient length of time to accurately quantify the average demand volume. Longer durations may not accurately represent the temporal changes in demand that occur over the course of the day or adequately replicate the
consequences of short term demand peaks on performance. Regardless, the volume (and other inputs) should always reflect an average for the analysis period.

The typical analysis period is 0.25 h. If the demand volume exceeds capacity for a given 0.25 h analysis period, then the analyst should consider evaluating: (1) a single long analysis period, or (2) consecutive 0.25 h periods. The multiple analysis periods, or the single long period, should start at time where there is no initial queue. Moreover, spillback should not occur during any analysis period. If it does, the analyst should consider using an alternative analysis tool that is able to model spillback conditions.

If a multi-period analysis is used, the analyst must separately evaluate each period and use the residual queue from one period as the initial queue for the next period. A multi-period analysis procedure is discussed in Chapter 16, Appendix F (Extension of Signal Delay Models to Incorporate the Effect of an Initial Queue). The analyst will have to modify or adapt this procedure if the boundary intersection operates as a roundabout or with stop control. Intersection and segment performance measures can be computed for each analysis period. The analyst must determine how to report these results since averaging across multiple analysis periods may obscure extreme values.

If a single long analysis period is used (such as 1 h), the analyst should use caution when interpreting the results because the adverse impact of short peaks in traffic demand may not be detected by this technique. Also, if the analysis period exceeds 0.25 h, the peak-hour factor should be set to 1.0.

**Mid-Segment Delay**

Through vehicles traveling along a segment can encounter a variety of situations that cause them to slow slightly, or even come to a stop. These encounters delay the through vehicle and cause the running time to increase. The situations that can cause this delay include:

- vehicles turning from the segment into an access point;
- pedestrians crossing at a mid-segment crosswalk;
- vehicles maneuvering into or out of an on-street parking space; and
- double-parked vehicles blocking a lane.

The delays due to vehicles turning from the segment are discussed more fully in the remainder of this section.

For operational analyses, a procedure is used to compute the delay due to vehicles turning left or right into an access point. The delay due to the other three sources in the list can be input to the methodology; however, they must be estimated by the analyst.

For planning and preliminary design analyses, the delay due to all four sources is input to the methodology. It can be estimated using the values listed in Exhibit 15-4. The values listed in this exhibit represent the delay due to turns, as averaged over all through vehicles traveling along the segment. Each value represents the delay incurred at one access point for a given volume and number of through lanes in the subject direction of travel. The total access-related delay incurred along the segment can be estimated as the product of the number of access points and the value from Exhibit 15-4.

EXHIBIT 15-4. Default Delays Due to Mid-Segment Access Points.

<table>
<thead>
<tr>
<th>Mid-Segment Volume, veh/h/ln</th>
<th>Through Vehicle Delay (s/veh/pt) by Number of Through Lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 Lane</td>
</tr>
<tr>
<td>200</td>
<td>0.04</td>
</tr>
<tr>
<td>300</td>
<td>0.08</td>
</tr>
<tr>
<td>400</td>
<td>0.12</td>
</tr>
<tr>
<td>500</td>
<td>0.18</td>
</tr>
<tr>
<td>600</td>
<td>0.27</td>
</tr>
<tr>
<td>700</td>
<td>0.39</td>
</tr>
</tbody>
</table>
The values listed in Exhibit 15-4 represent 10 percent left turns and 10 percent right turns from the segment at the access point. If the actual turn percentages are less than 10 percent then the delays can be reduced proportionally. For example, if the subject access point has 5 percent left turns and 5 percent right turns, then the values listed in the exhibit should be multiplied by 0.5 (= 5/10). Also, if a turn bay of adequate length is provided for one turn movement, then the values listed in the exhibit should be multiplied by 0.5. If both turn movements are provided a bay of adequate length, then the delay due to turns can be assumed to equal 0.0 s/veh/pt.

Geometric Design Data

This section describes the geometric design data listed in Exhibit 15-2. These data describe the geometric elements of the segment (or intersection) that influence traffic operation.

Number of Through Lanes on Segment

The number of through lanes on the segment represents the count of lanes that extend for the length of the segment and serve through vehicles. This is a “directional” value because it represents the number of through lanes serving the subject direction of travel along the segment. A lane provided for the exclusive use of turning vehicles is not included in this count.

Number of Lanes at Boundary Intersection

The number of lanes at the boundary intersection represents the count of lanes that are provided for each intersection traffic movement. For a turn movement, this count represents the lanes reserved for the exclusive use of turning vehicles. Turn movement lanes include exclusive turn lanes that extend for the length of the segment or lanes in a turn bay. If a turn movement shares a lane with a through movement, then the turn movement is indicated to have “0” lanes and the through movement is indicated to have the total number of lanes that serve both movements.

Number of Lanes at Mid-Segment Access Point

The number of lanes at a mid-segment access point represents the count of lanes that are provided for each access point traffic movement. The number of lanes input for each movement follows the same guidance as provided in the previous section for boundary intersections.

Turn Bay Length

The turn bay length represents the length of the bay for which the lanes have full width and queued vehicles can be stored. The turn bay length does not include the bay taper length. Bay length is measured parallel to the roadway centerline. It is needed for both the boundary intersections and the mid-segment access points. If a two-way-left-turn lane is provided for the left-turn movement at an access point, then the length entered should represent the effective storage length available to the subject left-turn movement (with consideration of adjacent access points and their associated left-turn vehicles that also store in the two-way left-turn lane).

Saturation Flow Rate

The saturation flow rate represents the maximum rate of flow for a through movement discharging from the queue at the stop line of a signalized intersection. For
planning or preliminary design analyses, this value can be estimated using established
default values or computed using the procedure described in Chapter 16. It should
reflect the effect of any turn movements that share the through lane.

For operational analyses, the procedure in Chapter 16 should be used to calculate
the input saturation flow rate. However, in this case, the adjustment factor for left turns
and that for right turns should both be set to 1.0. The effect of turn movements from a
shared lane is accounted for in the computational modules described in Appendix A.

**Segment Length**

The segment length represents the distance between the boundary intersections that
define the segment. The point of measurement at each intersection is the stop (or yield)
line (or its functional equivalent) in the subject direction of travel. This length is
measured along the centerline of the segment. It exceeds the “adjusted” segment length,
which is equal to the segment length minus the width of the upstream boundary
intersection.

**Intersection Width**

The intersection width applies only to the upstream boundary intersection. On a
two-way street, it represents the distance between the stop (or yield) line for the two
opposing segment through movements at the boundary intersection, as measured along
the centerline of the segment. On a one-way street, it represents the distance from the
stop line to the far side of the most distant traffic lane on the cross street.

**Restrictive Median Length**

The restrictive median length represents that portion of the adjusted segment length
that has a restrictive median (e.g., raised curb). This length is measured from median
nose to median nose and does not include the length of median openings.

**Non-Restrictive Median Length**

The non-restrictive median length represents that portion of the adjusted segment length
that has a non-restrictive median (e.g., two-way left-turn lane). This length is
measured from the start of the pavement markings that designate the non-restrictive
median to the end of these pavement markings. It does not include the length of median
openings.

**Percent Segment with Curb**

The percent of the segment with curb applies to the adjusted segment length and
represents that portion of this length that has curb along the right side of the segment. It
includes curb openings for driveways. This value is input for each direction of travel
along the segment.

**Number of Access Points Along Segment**

The number of access points along a segment represents the count of unsignalized
driveways and public street approaches located along the segment, regardless of the land
use served by the access point or its traffic demand. This number is counted separately
for the right-hand side of each segment travel direction and thus, represents a directional
input. This number must equal or exceed the number of active access points for which
delay to segment through vehicles is computed. Driveways that are unused can be
excluded.
Left-Turn Phasing Data

This section describes the left-turn phasing data listed in Exhibit 15-2. These data describe the signal phase sequence at a signalized boundary intersection. The urban streets methodology is based on the assumption that the segment through movements are served by the coordinated phases.

Operational Mode

The operational mode describes the manner in which the left-turn movement is served by the controller. The three modes are: permissive, protected, and protected-permissive. These modes are described in Chapter 10. In general, the operational mode used for one left-turn movement is also used for the opposing left-turn movement. However, this agreement is not required.

Phase Sequence

The phase sequence describes the order of service provided to left-turn movements by the signal controller, relative to the other intersection movements. The typical options include: no left-turn phase (permissive-only), leading left-turn, lagging left-turn, and split. These options are defined in Chapter 10. When using the urban streets methodology, split phasing should be selected for a one-way street.

Controller Settings

This section describes the controller setting data listed in Exhibit 15-2. These data describe the traffic control signal operation at a signalized boundary intersection.

Minimum Green

The minimum green setting represents the least amount of time that a green signal indication will be displayed for a movement. It is input for each non-coordinated signal phase. Its duration is based on considerations of driver reaction time, queue length, and driver expectancy. Minimum green may be as short as 3 s for a left-turn phase and 5 to 10 s for a through movement. For intersections without pedestrian push buttons, the minimum green interval may also need to be sufficiently long to allow a pedestrian to cross during the concurrent vehicular phase.

Yellow + Red Clearance

The yellow-plus-red-clearance input represents the sum of the yellow change interval and the red clearance interval. It is input for each signal phase. The yellow change interval is intended to alert a driver to the impending presentation of a red indication. It ranges in duration from 3 to 6 s with longer values in this range used with phases serving high speed movements. The red clearance interval can be used to allow a brief period of time to elapse following the yellow indication and during which the signal heads associated with the ending phase and all conflicting phases display a red indication. If used, the red clearance interval is typically 1 or 2 s in duration.

Passage Time

Passage time is the maximum amount of time one vehicle actuation can extend the green interval when green is displayed. It is input for each non-coordinated signal phase. It is also referred to as vehicle interval, extension interval, extension, or unit extension.
The passage timer starts to time from the instant the vehicle actuation is removed. A subsequent actuation will reset the passage timer. When the passage timer reaches the passage time setting and there is an actuation on a conflicting phase, the phase will terminate by gap out.

Passage time values are typically chosen based on consideration of detection zone length, detection zone location (relative to the stop line), number of lanes served by the phase, and vehicle speed. Longer passage times are often used with shorter detection zones, greater distance between the zone and stop line, fewer lanes, and slower speeds.

The objective when determining the passage time value is to make it large enough to ensure that all queued vehicles are served but to not make it so large that it extends for randomly arriving traffic. On high-speed approaches, this objective is broadened to include not making the passage time so large that the phase frequently extends to its maximum setting (i.e., maxes-out) such that safe phase termination is compromised.

**Cycle Length**

The cycle length is the time elapsed between the “end time” of two sequential presentations of a coordinated phase. Phase end time is defined as the end of the green interval associated with the phase. Cycle length is input for the intersection.

**Phase Recall**

If used, recall causes the controller to place a call for a specified phase each time the controller is servicing a conflicting phase. It is input for each non-coordinated signal phase. There are two types of recalls modeled in the phase duration procedure: minimum recall and maximum recall.

Invoking minimum recall causes the controller to place a continuous call for vehicle service on the phase and, when it is displaying its green interval, service the phase until its minimum green interval times out. The phase can be extended if actuations are received.

Invoking maximum recall causes the controller to place a continuous call for the vehicle service on the phase. It results in the presentation of the green indication for its maximum duration every cycle. In coordinated-actuated operation, the maximum phase duration is limited by the force-off point. This point is established using the input phase splits and force mode (as described in a subsequent paragraph). Using maximum recall on all non-coordinated phases yields an equivalent pretimed operation.

**Ring Entry Mode**

The entry mode describes the manner in which the controller selects phases to display in each ring. Two entry modes are possible: dual entry and single entry. This mode is input for each signal phase.

A phase operating in dual entry is available to be called by the controller, even if no actuations have been received for this phase. A phase operating in single entry will only be called if actuations have been received.

When timing a cycle, a point is reached where the next phase (or phases) to be timed is on the other side of a barrier. At this point, the controller will check the phases in each ring and determine which to call. If a call does not exist in a ring, a phase designated as dual entry in that ring will be called by the controller in a predetermined manner.

**Simultaneous Gap-Out**

Simultaneous gap-out describes the manner in which the controller terminates a phase when it ends at a barrier. This mode is either enabled or disabled. It is a phase-
specific setting; however, it is typically set the same for all phases that serve the same street.

Simultaneous gap-out dictates controller operation when a barrier must be crossed to serve the next call and one phase is active in each ring. If simultaneous gap-out is enabled, it requires that both phases reach a point of being committed to terminate (via gap out, max out, or force-off) at the same time.

If simultaneous gap-out is disabled, one phase can reach this point first (but remain green) before the other phase. In this situation, the first phase to commit to termination does not change its status while waiting for the other phase to commit to termination. Regardless of which mode is in effect, the barrier is not crossed until both phases are committed to terminate.

**Phase Splits**

Each non-coordinated phase is provided a “split” time. This time represents the sum of the green, yellow change, and red clearance intervals for the phase. The rationale for determining the green interval duration varies among agencies; however, it is often related to the optimum pretimed green duration. This optimum value is obtained from a signal timing optimization software program that is used to model the subject segment. The phase split for each phase in a concurrent pair (i.e., 1 and 5, 3 and 7, 4 and 8) is typically set at the same value. This equality is not a requirement for modern controllers; however, the phase duration procedure was developed based on the assumption that concurrent phase pairs would have the same split.

**Offset and Reference Point**

The reference phase is specified to be one of the two coordinated phases (i.e., 2 or 6). The offset entered in the controller represents the time that the reference phase begins (or ends) relative to the system master time zero. The offset must be specified as being referenced to the beginning, or the end, of the green interval of the reference phase. The offset reference point is typically the same at all intersections in a given signal system.

**Force Mode**

This mode is a controller-specific setting. It is set to “fixed” or “floating.” The controller calculates the phase force-off point for each non-coordinated phase based on the force mode and the phase splits. When set to the fixed mode, each non-coordinated phase has its force-off point set at a fixed time in the cycle, relative to time zero on the system master. This operation allows unused split time to revert to the following phase. When set to the floating mode, each non-coordinated phase has its force-off point set at the split time after the phase first becomes active. This operation allows unused split time to revert to the coordinated phase (referred to as an “early return to green”).

**Green-to-Cycle Length Ratio for Segment Through Phase**

The green-to-cycle length ratio \( \frac{g}{C} \) represents the proportion of the cycle that is effectively green for the segment through movement. This ratio is input for planning and preliminary design analyses. A procedure is provided in the methodology for computing this ratio for the operational analysis of a coordinated system. This procedure is described in Appendix A.

For the operational analysis of a non-coordinated system, the green-to-cycle length ratio is an input for a pretimed intersection. In contrast, the procedure described in Appendix B of Chapter 16 is used to compute this ratio for a fully-actuated intersection. In this latter situation, the computed ratio represents an average for the analysis period.
Dallas Phasing

The Dallas phasing option describes the signal indication sequence used to control the left-turn movements operating in a protected-permissive mode. This option is (or is not) invoked for both left-turn opposing movements on a given street. The phase duration procedure does not recognize the option of invoking it for only one of the two left-turn movements on a given street. The Dallas phasing option eliminates the yellow trap problem (see Chapter 10) and can increase the capacity of the left-turn movements. However, the left-turn display must be modified and the indications powered using a controller overlap.

Other Data

Stop Line Detector Length

The stop line detector length represents the length of the detection zone used to extend the green indication for queue service. This detection zone is typically located near the stop line and may have a length of 40 ft or more. However, it can be located some distance upstream of the stop line and be as short as 6 ft in length. This latter configuration typically requires a long minimum green or use of the controller’s variable initial setting.

Speed Limit

Speed limit is used in the methodology as a general descriptor of the street environment and its geometric design. In this sense, speed limit is highly correlated with the environmental and geometric factors that have a direct influence on driver speed choice. As such, it represents a single input variable that is highly correlated with operating speed and is used as a convenient way to limit the need for numerous environmental and geometric input data.

The convenience of using speed limit as an input variable comes with a caution—the analyst must not infer a cause-and-effect relationship between the input speed limit and the estimated segment performance measures. More specifically, the computed change in performance resulting from a change in the input speed limit is not likely to be indicative of performance changes that will actually be realized. Research indicates that a change in speed limit has very little effect on the actual operating speed.

The methodology is based on the assumption that the posted speed limit is:
(1) consistent with that found on other streets that are similar to the subject segment and
(2) consistent with agency policy regarding specification of speed limits. If it is known that the posted speed limit does not satisfy these assumptions, then the speed input to the procedure should be adjusted such that this consistency is achieved.

Determining Traffic Demand Adjustments

This section describes the traffic demand adjustments associated with the operational analysis of a coordinated signal system. It also discusses the need for an origin-destination matrix for the subject segment. Finally, a check for spillback occurrence is summarized. These adjustments and checks are not applied in planning and preliminary design analyses nor in the operational analysis of a non-coordinated street segment. Procedures for making these adjustments in the operational analysis of a coordinated street are described in Appendix A.
Capacity Constraint

When the volume for an intersection traffic movement exceeds its capacity, the volume discharging from the intersection is restricted (or metered). When this metering occurs for a movement that enters the subject segment, the volume arriving at the downstream signal is reduced below the unrestricted value.

To determine if metering occurs, the capacity of each upstream movement that discharges into the subject segment must be computed and checked with the associated demand volume. If this volume exceeds movement capacity, then the volume entering the segment must be reduced to equal the movement capacity.

Volume Balance

Volume balance describes a condition where the combined volume from all movements entering a segment equals the combined volume exiting the segment, in a given direction of travel. The segment is “completely” balanced when volume balance exists in both directions of travel. Unbalanced volumes often exist in turn movement counts when the count at one intersection is taken at a different time than the count at the adjacent intersection. They are very likely to exist when turn movement counts are not collected at mid-segment access points.

The accuracy of the performance evaluation will be adversely impacted if the volumes are not balanced. The extent of the impact is based on the degree to which the volumes are unequal. To balance the volumes, the methodology assumes that the volumes entering the segment are correct and adjusts the exiting volumes in a proportional manner such that a balance is achieved.

Origin-Destination Distribution

The volume of traffic that arrives at a downstream intersection for a given downstream movement represents the combined volume from each upstream point of entry weighted by its percentage contribution to the downstream movement. The distribution of these contribution percentages between each upstream and downstream pair is represented as an origin-destination distribution matrix.

The concept of an origin-destination distribution matrix is illustrated by example. Consider the street segment shown in Exhibit 15-5. There are three entry volumes at the upstream intersection that contribute to the three exit volumes at the downstream intersection. There is also an entrance and exit volume at the access point located between the two intersections. It should be noted that 1350 veh/h enter the segment and 1350 veh/h exit the segment, thus there is volume balance for this sample segment. The origin-destination distribution matrix for this sample street segment is shown in Exhibit 15-6.

The column totals in the last row of Exhibit 15-6 correspond to the entry volumes shown in Exhibit 15-5. The row totals in the last column of Exhibit 15-6 indicate the exit volumes. The individual cell values indicate the volume contribution of each upstream movement to each downstream movement. For example, of the 1000 through vehicles that enter the segment, 877 depart the segment as a through movement, 46 depart as a left-turn movement, and so on. The volumes in the individual cells are sometimes expressed as a proportion of the column total.

The urban streets methodology computes an origin-destination matrix for each segment junction (i.e., intersection or access point) that is downstream of the upstream signalized intersection. The matrix associated with the downstream intersection is used to compute the proportion of vehicles arriving during the green indication for each exit movement. The matrix associated with a downstream access point is used to compute the proportion of time that a platoon is passing through the access point and effectively blocking non-priority movements from entering or crossing the street.
EXHIBIT 15-5. Entry and Exit Volume on Sample Street Segment.


<table>
<thead>
<tr>
<th>Origin Volume by Movement, veh/h</th>
<th>Destination Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Movement</td>
</tr>
<tr>
<td>Left</td>
<td>Through</td>
</tr>
<tr>
<td>2</td>
<td>46</td>
</tr>
<tr>
<td>188</td>
<td>877</td>
</tr>
<tr>
<td>3</td>
<td>36</td>
</tr>
<tr>
<td>7</td>
<td>41</td>
</tr>
<tr>
<td>200</td>
<td>1000</td>
</tr>
</tbody>
</table>

**Spillback Occurrence**

Segment spillback can be characterized as one of two types: cyclic and sustained. Cyclic spillback occurs when the queue from the downstream intersection backs into the upstream intersection as a result of queue growth during the red indication. When the green indication is presented, the queue dissipates and spillback is no longer present for the remainder of the cycle. This type of spillback can occur on short street segments with relatively long signal cycle lengths.

Sustained spillback occurs at some point during the analysis period and is a result of oversaturation (i.e., more vehicles discharging from the upstream intersection than can be served at the subject downstream intersection). The queue does not dissipate at the end of each cycle. Rather, it remains present until the downstream capacity is increased or the upstream demand is reduced.

Use of the urban streets methodology to evaluate street segments with significant, sustained spillback is problematic because of unsteady conditions and complex interactions. The methodology computes the time when sustained spillback occurs, if it occurs. If this time of occurrence is shorter than the analysis period, then the methodology may not yield accurate performance estimates. In this situation, the analyst should consider reducing the analysis period such that it ends before spillback occurs or consider the use of an analysis tool that more accurately models spillback events.

**DETERMINING RUNNING TIME**

There are two principal components of the time that a vehicle takes to travel the length of a street segment, they are: (1) segment running time, and (2) control delay at
the downstream signalized intersection. The factors that contribute to segment running time are the subject of this section. They are identified in the following equation:

\[ T_R = \frac{6.0 - l_1}{0.0025 L} + \frac{3600 L}{5280 S_f} f_v + \sum_{i=1}^{N_a} d_{ap,i} + d_{other} \]  

(15-2)

where,

- \( T_R \) = segment running time, s;
- \( l_1 \) = start-up lost time, s;
- \( L \) = segment length, ft;
- \( S_f \) = free-flow speed, mph;
- \( f_v \) = proximity adjustment factor;
- \( d_{ap,i} \) = delay due to left or right-turns from the street into access point \( i \), s/veh;
- \( N_a \) = number of access points along the subject segment, approaches; and
- \( d_{other} \) = delay due to other sources along the segment (e.g., curb parking, pedestrians, bicyclists, etc.), s/veh.

The first term in Equation 15-2 accounts for the time required to accelerate to the running speed, less the start-up lost time used to compute the through movement delay. The divisor in this term is an empirical adjustment that minimizes the contribution of this term for longer segments. It partially reflects a tendency for drivers to offset this added time by adopting slightly higher mid-segment speeds than reflected in the start-up lost time estimate. This term is only applicable to segments with signalized or stop-controlled boundary intersections. The subsequent parts of this section describe the other components of Equation 15-2.

**Free-Flow Speed**

Free-flow speed represents the average speed of through automobile drivers when traveling along a street under low-volume conditions and when not delayed by any control device or other vehicle. It reflects the effect of the street environment on driver speed choice. Elements of the street environment that may influence driver speed choice under free-flow conditions include speed limit, access point density, median type, curb presence, and segment length. Free-flow speed is computed as:

\[ S_f = S_{f0} f_L \]  

(15-3)

where,

- \( S_{f0} \) = base free-flow speed, mph; and
- \( f_L \) = segment length adjustment factor.

The first term of Equation 15-3 represents the base free-flow speed. The second term represents an adjustment to this speed to account for the effect of segment length.

**Base Free-Flow Speed**

The base free-flow speed is defined to be the free-flow speed on longer street segments. It includes the influence of speed limit, access point density, median type, and curb presence. It is computed using the following equation:

\[ S_{f0} = S_0 + f_{CS} + f_A \]  

(15-4)

where,

- \( S_0 \) = speed constant, mph;
- \( f_{CS} \) = adjustment for cross section (see Exhibit 15-7), mph; and
- \( f_A \) = adjustment for access points (see Exhibit 15-7), mph.
The speed constant and adjustment factors used in Equation 15-4 are listed in Exhibit 15-7. Equations provided in the table footnote can also be used to compute these adjustment factors.


<table>
<thead>
<tr>
<th>Speed Limit, mph</th>
<th>Median Type</th>
<th>Percent with Restrictive Median, %</th>
<th>Adjustment for Cross Section (f_{cs}), mph</th>
<th>No Curb</th>
<th>Curb</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>Restrictive</td>
<td>20</td>
<td>0.3</td>
<td>-0.9</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>Restrictive</td>
<td>40</td>
<td>0.6</td>
<td>-1.4</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>Restrictive</td>
<td>60</td>
<td>0.9</td>
<td>-1.8</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>Restrictive</td>
<td>80</td>
<td>1.2</td>
<td>-2.2</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>Restrictive</td>
<td>100</td>
<td>1.5</td>
<td>-2.7</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>Non-Restrict.</td>
<td>not applicable</td>
<td>0.0</td>
<td>0.0</td>
<td>-0.5</td>
</tr>
<tr>
<td>55</td>
<td>Non-Restrict.</td>
<td>not applicable</td>
<td>0.0</td>
<td>-0.5</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Access Density (D_a), points/mi</th>
<th>Adjustment for Access Points (f_a) by Lanes (N), mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Lanes</td>
<td>2 Lanes</td>
</tr>
<tr>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>-0.2</td>
</tr>
<tr>
<td>4</td>
<td>-0.3</td>
</tr>
<tr>
<td>10</td>
<td>-0.8</td>
</tr>
<tr>
<td>20</td>
<td>-1.6</td>
</tr>
<tr>
<td>40</td>
<td>-3.1</td>
</tr>
<tr>
<td>60</td>
<td>-4.7</td>
</tr>
</tbody>
</table>

Notes:
1. \( S_\beta = 25.6 + 0.47 \cdot S_{pl} \), where \( S_{pl} \) = speed limit, mph.
2. \( f_a = 0.015 \cdot P_{m} - 0.0047 \cdot P_{curb} - 0.00037 \cdot P_{curb} \cdot P_{m} \), where, \( P_{m} \) = percent of segment length with restrictive median; and \( P_{curb} \) = percent of segment with curb on the right-hand side.
3. \( f_a = -0.078 \cdot D_a/N \) with, \( D_a = D_{a,sub} + D_{a,other} \), where, \( D_{a,sub} \) = access point density on right side in subject direction of travel, points/m; \( D_{a,other} \) = access point density on right side of other direction of travel; and \( N \) = number of through lanes on the segment in the subject direction of travel.

### Adjustment for Segment Length

Empiric evidence suggests that a “short” segment length tends to influence a driver’s choice of free-flow speed. Shorter segments have been found to have a slower free-flow speed, all other factors being the same. The adjustment factor in Equation 15-3 accounts for this influence. The following equation is used to compute the value of this factor.

\[
f_L = 1.02 - 4.7 \frac{S_\beta}{L} - 19.5 \leq 1.0 \tag{15-5}
\]

If segment length is less than 400 ft, then \( L \) should be set to 400 ft to use this equation.

### Adjustment for Vehicle Proximity

The proximity adjustment factor adjusts the free-flow running time to account for the effect of traffic density. The adjustment results in an increase in running time (and corresponding reduction in speed) with an increase in volume. The reduction in speed is a result of shorter headways associated with the higher volume and the driver’s propensity to be more cautious when headways are short. The following equation is used to compute the proximity adjustment factor.
where,  

\[ v = \text{mid-segment volume, veh/h; and} \]

\[ N = \text{number of through lanes for length of segment, ln.} \]

The relationship between running speed and volume for an urban street segment is shown in Exhibit 15-8. Trend lines are shown for three specific free-flow speeds. Each trend line shows a reduction of about 2.5 mph at a lane flow rate of 1000 veh/h/ln, relative to the free-flow speed. The trend lines are extended beyond 1000 veh/h/ln in the exhibit; however, it is unlikely that a volume in excess of this amount will be experienced on a segment bounded by intersections at which the through movement is regulated by a traffic control device.

**EXHIBIT 15-8. Speed-Flow Relationship for Urban Street Segments.**

![Graph showing speed-flow relationship](image)

### Additional Running Time Due to Mid-Segment Delay Sources

Vehicles turning from the subject street segment into an access point can cause a delay to following through vehicles. For right-turn vehicles, the delay results when the following vehicles' speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane.

On an undivided street, the magnitude of the delay due to turning vehicles for a typical access point can range from 0.04 to 0.72 s/veh/pt, as shown in Exhibit 15-4. Although this delay is relatively small, it can accumulate to a significant level if there are many access points along the segment.

There are numerous other factors that could cause a driver to reduce speed or to incur delay when traveling along a segment. For example, a vehicle that is completing a parallel parking maneuver may cause following vehicles to incur some delay. Also, vehicles that yield to pedestrians at a mid-segment crosswalk may incur delay. Finally,
bicyclists riding in a traffic lane (or in an adjacent bike lane) may directly, or indirectly, cause vehicular traffic to adopt a lower speed.

**DETERMINING PROPORTION ARRIVING DURING GREEN**

Control delay and stop rate at a signalized intersection are highly dependent on the proportion of vehicles that arrive during the green and the red signal indications. Delay and stop rate are reduced when a larger proportion of vehicles arrive during the green indication. The proportion of arrivals during the green indication can be measured in the field by dividing the count of vehicles that arrive to the intersection stop line (or back of queue, if present) during the green indication by the total count of vehicles that arrive to the intersection.

For an operational analysis of coordinated systems, the methodology includes a procedure for computing the proportion of vehicles that arrive during the green associated with the segment through and left-turn phases (i.e., the internal lane groups). This procedure is described in Appendix A. For phases associated with external lane groups, the following equation is used to compute the proportion of vehicles arriving during green.

\[
P_i = \frac{P \cdot g}{C}
\]

where,
- \(P\) = proportion of vehicles arriving during green;
- \(g\) = effective green time, s; and
- \(C\) = cycle length, s.

For planning and preliminary design analyses of coordinated systems, Equation 15-7 is used for the segment through lane group. For non-coordinated systems, Equation 15-7 is used with platoon ratio of 1.0.

The value obtained from Equation 15-7 may not exceed 1.0. The proportion of vehicles arriving during green should reflect the arrival pattern of the traffic movements that comprise the lane group. If this group includes two or more movements, then Equation 15-7 used to compute \(P_i\) for each movement, based on an estimate of each movement’s platoon ratio. The value of \(P\) for the lane group would then be computed as a weighted average of the movement \(P_i\), where the weight is the volume of each movement.

**DETERMINING SIGNAL PHASE DURATION**

A desirable feature of coordinated-actuated signal operation is its ability to adjust the green interval duration to cyclic variation in traffic demand. Although an actuated phase may vary in duration on a cycle by cycle basis, it can be adequately represented by its long-run average duration for the purpose of operational evaluation. Specifically, this average duration can be used to estimate the delay or stops incurred by traffic movements served by the phase.

This section describes the concepts associated with the estimation of phase duration for a coordinated-actuated intersection. These concepts are incorporated in the procedure for computing phase duration as part of an operational analysis. This procedure is described in more detail in Appendix A.

For planning and preliminary design analyses, the green-to-cycle-length ratio for the segment through phase is provided as an input, so the phase duration procedure is not used. This ratio is also input for the operational analysis of a pretimed intersection that is part of a non-coordinated system. The procedure in Appendix B of Chapter 16 is used to compute the average green-to-cycle-length ratio for the operational analysis of an actuated intersection that is part of a non-coordinated signal system.
Coordinated Phase Duration

There are two phase types at a coordinated-actuated intersection: coordinated and non-coordinated. The duration of a coordinated phase is dictated by the cycle length and the force-off settings for the non-coordinated phases. These settings define the points in the signal cycle at which each non-coordinated phase must end. The force-off settings are used to ensure that the coordinated phases receive a green indication at a specific time in the cycle. Presumably, this time is synchronized with the coordinated phase time at the adjacent intersections such that traffic is progressed along the street segment. In general, the duration of a coordinated phase is equal to the cycle length less the time allocated to the conflicting phase in the same ring and the time allocated to the minor street phases. Detectors are not typically assigned to the coordinated phase and this phase is not typically extended by the through vehicles it serves.

Non-Coordinated Phase Duration

The duration of a non-coordinated phase is dictated by traffic demand in much the same manner as is an actuated phase. However, the non-coordinated phase duration is typically constrained only by its force-off setting (rather than a maximum green setting). A non-coordinated phase is also referred to herein as an “actuated” phase when it is clear that the context is with regard to coordinated-actuated operation.

Actuated Phase Duration

The duration of an actuated phase is comprised of four time periods. The first period represents the time lost while the queue reacts to the signal indication changing to green. The second interval represents the time required to clear the queue of vehicles. The third period represents the time the green is extended by randomly arriving vehicles. It ends when there is a gap in traffic (i.e., gap out) or the green extends to the maximum limit (i.e., max out). The last period represents the change period. The duration of an actuated phase can be expressed by the following equation:

\[ D_p = l_1 + g_s + g_e + Y \]  \hspace{2cm} (15-8)

where,
- \( D_p \) = phase duration, s;
- \( l_1 \) = start-up lost time, s;
- \( g_s \) = queue service time, s;
- \( g_e \) = green extension time, s; and
- \( Y \) = combined yellow change and red clearance interval duration (intergreen), s.

The relationship between the variables in Equation 15-8 is shown in Exhibit 15-9 using a queue accumulation polygon. Also shown in this exhibit is the relationship between the equation variables and the queue length during the average signal cycle. During the red interval, vehicles arrive and form a queue. The queue reaches its maximum size \( l_1 \) seconds after the red interval ends. At this time, the queue begins to discharge at a rate equal to the saturation flow rate \( s \) less the arrival rate during green \( q_c \). The queue clears \( g_s \) seconds after it first begins to discharge. Thereafter, random vehicle arrivals to the intersection are detected and extend the green interval. Eventually, a gap occurs in traffic (or the maximum green limit is reached) and the green interval ends. The end of the green interval coincides with the end of the extension time \( g_e \).
The calculation of control delay and stop rate for an actuated phase is based on the phase effective green time. This time is computed with the following equation:

\[ g = g_e + g_s + e \]  

(15-9)

where,

\[ e = \text{extension of effective green, s.} \]

**Detection Design**

A non-coordinated (or actuated) phase is extended through the detection of vehicles on the intersection approach. This detection is typically provided by one or more inductive loop detectors embedded in the pavement.

The detector layout in a traffic lane and its associated passage time define a limiting time headway between successive vehicle activations (i.e., calls) that dictate when the controlling signal phase can end. This limiting headway is referred to as the maximum allowable headway (MAH). Call headways arriving to the controller at intervals shorter than the MAH will extend the green interval. The first call headway arriving at an interval longer than the MAH will result in gap out.

The relationship between passage time PT, detection zone length Ld, vehicle length Lv, average running speed Sa, and MAH is shown in Exhibit 15-10. The two vehicles shown are traveling from left to right and have a headway that is just equal to the MAH such that the second vehicle just arrives to the detector at the instant the passage time is set to expire.
MAH values that are too long or too short can compromise the efficiency of intersection operation. Exceptionally short MAH values tend to increase the frequency of premature gap out and the delay to unserved vehicles. Exceptionally long MAH values may allow low-volume movements to unnecessarily extend the green and delay vehicles served by other phases. The optimum detection design is one that yields a MAH that is short enough to ensure a “snappy” operation but not so short that premature gap outs occur. MAH’s that are typically found to be most effective for stop line detection range from 2.5 to 3.5 s.

Exhibit 15-11 identifies effective combinations of passage time and detection zone length for a range of zone lengths and approach speeds. In general, a long detection zone located at the stop line is the most efficient at identifying when the queue has cleared. However, a 6 ft detector located 3.5 s travel time upstream of the stop line is sometimes used with volume-density features to maximize detector life and eliminate unneeded extension once the last queued vehicle clears the stop line.

**EXHIBIT 15-11. Effective Combinations of Passage Time and Detector Length.**

<table>
<thead>
<tr>
<th>Speed Limit, mph</th>
<th>Passage Time by Detection Zone Length and MAH, 1 s</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6 ft Detection</td>
</tr>
<tr>
<td></td>
<td>3.5 s MAH</td>
</tr>
<tr>
<td>30</td>
<td>3.0</td>
</tr>
<tr>
<td>35</td>
<td>3.0</td>
</tr>
<tr>
<td>40</td>
<td>3.0</td>
</tr>
<tr>
<td>45</td>
<td>3.0</td>
</tr>
<tr>
<td>50</td>
<td>3.0</td>
</tr>
<tr>
<td>55</td>
<td>3.0</td>
</tr>
</tbody>
</table>

Note:
1. \( PT = MAH - (L_d + L_v)(1.47S_a) \); where, \( PT = \) passage time, s; \( MAH = \) maximum allowable headway (= 3.0 s); \( L_d = \) length of the detection zone, ft; \( L_v = \) detected length of vehicle (=17 ft); \( S_a = \) average speed (= 0.95 \([25.6 + 0.47 S_p]\) ), mph; and \( S_p = \) speed limit, mph.
2. Represents a single presence-mode detector located 3.5 s travel time in advance of the stop line with (1) gap reduction using a 2.0 s minimum gap and (2) variable initial interval.

**DETERMINING THROUGH CONTROL DELAY**

The control delay incurred by through vehicles as they exit the segment is the basis for travel time estimation. Ideally, this delay is computed for just those vehicles that enter and exit the segment as a through vehicle. However, the nature of the delay models used with this methodology make it difficult to separate the delay to through vehicles from the delay to non-through vehicles. A reasonable approximation of delay incurred by through vehicles is the delay computed for the through lane group at the boundary intersection.

The type of control used at the boundary intersection will have a significant influence on through delay. Procedures for computing this delay at intersections with the following control types are described in this manual:

- all-way stop control (Chapter 17);
- yield control at a roundabout intersection (Chapter X);
- non-coordinated signal control (Chapter 16); and
- coordinated signal control (Chapter 16).

The analyst should use the procedure in the appropriate chapter to estimate the through control delay for the boundary intersection.

The first three types of control in the aforementioned list are applicable to non-coordinated systems. As such, the application of the control delay procedure should be based on “isolated” operation. For example, the delay estimation procedure in
Chapter 16 - Signalized Intersections has a sensitivity to arrival type. When used to estimate through delay in a non-coordinated system, the arrival type should be specified as “3” (i.e., random arrivals).

For signalized intersections in a coordinated system, the delay estimation procedure in Chapter 16 is also used to estimate through delay. However, for this application, some extensions have been added to the Chapter 16 procedure to adapt it to coordinated flow conditions. These extensions are described in the remainder of this section.

The procedure described in this section assumes that the through lane group is not adversely affected by turn movements that may share the through lanes. This assumption is reasonable for planning and preliminary design analyses. However, it may not be appropriate for an operational analysis. A procedure is included in the methodology for computing through lane group delay when a shared lane exists. This procedure is described in Appendix A.

**Delay Equation**

The following equation is used to estimate the delay to the through lane group:

\[
d = d_1 (PF) + d_2 + d_3
\]

\[d_1 = \frac{0.5 C (1 - g/C)^2}{1 - \min(1, X) g/C}
\]

\[d_2 = 900 T \left( X_a - 1 \right) + \sqrt{(X_a - 1)^2 + \frac{8 k I X_a}{c_a T}}
\]

where,
- \(d\) = control delay, s/veh;
- \(d_1\) = uniform delay, s/veh;
- \(d_2\) = incremental delay, s/veh;
- \(d_3\) = initial queue delay, s/veh;
- PF = progression adjustment factor;
- \(c\) = capacity (= \(N s g/C\)), veh/h;
- \(c_a\) = available capacity, veh/h;
- \(s\) = saturation flow rate, veh/h/ln;
- \(N\) = number of lanes, ln;
- \(X\) = volume-to-capacity ratio (= \(v/c\));
- \(X_a\) = volume-to-available-capacity ratio (= \(v/c_a\));
- \(v\) = volume, veh/h;
- \(T\) = duration of analysis period, h;
- \(k\) = incremental delay calibration factor; and
- \(I\) = upstream filtering adjustment factor.

These three equations are described in more detail in Chapter 16. This chapter also provides a procedure for computing saturation flow rate. The product \(d_1 PF\) is referred to hereafter as the “deterministic” delay component.

The incremental delay calibration factor is used to account for the ability of the detection design to minimize green extension due to random arrivals. Values for this factor vary from 0.04 to 0.50, depending on the controller passage time and lane-group volume-to-capacity ratio. Exhibit 16-13 in Chapter 16 lists the recommended factor values. This exhibit indicates that a factor value of 0.5 is appropriate for the segment through lane group in a coordinated system. A default factor value of 0.4 is appropriate for planning or preliminary design analyses of non-coordinated movements.
Progression Adjustment Factor

The progression adjustment factor is used to adjust the delay estimate based on the quality of traffic progression. The adjustment is applied to the uniform delay term in Equation 15-10 because progression has a direct effect on arrival patterns. This factor applies to all coordinated lane groups, regardless of their type of control. It is computed using Equation 15-13.

\[
P_F = \frac{1 - P}{(1 - g/C)^2} (1 - y) \left( \frac{1 - g/C + y(1 - P C/g)}{1 - y P C/g} \right)
\]

(15-13)

where,

\[
y = \text{flow ratio} = \min[1, X g/C].
\]

Upstream Filtering Adjustment Factor

The upstream filtering adjustment factor \(I\) in Equation 15-12 accounts for the effects of filtered arrivals from upstream signals. It reflects the way that upstream signals decrease the variance in the number of arrivals per cycle at the subject (i.e., downstream) signalized intersection. The value of \(I\) ranges from 0.09 to 1.00. Lower values in this range correspond to lower incremental delay. A value of 1.0 is appropriate for an isolated intersection (i.e., one that is one mile or more from the nearest upstream signalized intersection). A value of less than 1.0 is appropriate for non-isolated intersections.

Exhibit 15-12 lists values for \(I\) for non-isolated intersections. The value of \(I\) in this exhibit are based on \(X_u\), the weighted volume-to-capacity ratio of all upstream movements contributing to the volume in the subject intersection lane group. This ratio is computed as a weighted average with the volume-to-capacity ratio of each contributing upstream movement weighted by its discharge volume. For planning and preliminary design analyses, \(X_u\) can be approximated as the volume-to-capacity ratio of the through lane group at the upstream signalized intersection that enters the subject segment.

<table>
<thead>
<tr>
<th>EXHIBIT 15-12. Upstream Filtering Adjustment Factor.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream Volume-to-Capacity Ratio ((X_u))</td>
</tr>
<tr>
<td>0.40 0.50 0.60 0.70 0.80 0.90 1.0</td>
</tr>
<tr>
<td>Filtering Adj. Factor ((I)) 0.922 0.858 0.769 0.650 0.500 0.314 0.090</td>
</tr>
<tr>
<td>Note: (I = 1.0 - 0.91 X_u^{0.68}) and (X_u &lt; 1.0)</td>
</tr>
</tbody>
</table>

Available Capacity

Equation 15-12 is derived from a queueing theory model that assumes a constant capacity (i.e., service time). For this reason, it is appropriate for modeling overflow queues at a pretimed intersection with fixed phase duration. However, simulation data indicate that Equation 15-12 tends to overestimate the overflow queue for an actuated phase when the average phase duration is used to estimate lane group capacity. Unlike a pretimed phase, an actuated phase has an “adaptive” capacity because it maintains the green indication until the queue is served on a cycle-by-cycle basis, regardless of how the demand may vary from cycle to cycle. Only when the actuated phase reaches its maximum green limit can an overflow possibly occur. Thus, an actuated phase minimizes the occurrence of an overflow queue (i.e., cycle failure).
To account for the ability of an actuated phase to prevent overflow, the capacity used in Equation 15-12 should be based on the maximum green limit. This calculation is shown in the following equation:

\[ c_a = \frac{g_a s N}{C} \]  

(15-14)

with

\[ g_a = G_{\text{max},e} + \bar{Y} - l_1 - l_2 \]  

(15-15)

where,

- \( c_a \) = available capacity, veh/h;
- \( g_a \) = available effective green time, s;
- \( G_{\text{max},e} \) = equivalent maximum green duration, s;
- \( l_1 \) = start-up lost time, s;
- \( l_2 \) = end lost time, s (= \( \bar{Y} - e \)); and
- \( \bar{Y} \) = combined yellow change and red clearance interval duration (intergreen), s.

Equation 15-14 is used to estimate the available capacity for any non-coordinated (or actuated) phases of interest. For coordinated phases or pretimed phases, the available effective green time \( g_a \) is equal to the effective green time \( g \) and the available capacity \( c_a \) is equal to the capacity \( c \).

For the operational analysis of a coordinated system, a procedure is provided in the methodology to estimate the equivalent maximum green duration. This procedure is described in Appendix A. For planning and preliminary design analyses where the segment through phase is coordinated, the following equation is used to estimate the available effective green time.

\[ g_a = C \frac{(g/C)}{C} \]  

(15-16)

where,

- \( g_a \) = available effective green time, s; and
- \( g/C \) = input \( g/C \) ratio for the through phase.

Equation 15-16 is also used when the segment through phase is pretimed.

**DETERMINING THROUGH STOP RATE**

This section describes a procedure for computing stop rate for the through movement at a signalized intersection. Stop rate is a useful performance measure for evaluating coordinated signal systems, given that such systems are intended to maintain traffic progression and minimize stops.

As with control delay, stop rate is most meaningful when used to describe vehicles that enter and exit the segment as a through vehicle. However, the nature of the stop rate models used with this methodology make it difficult to separate the stops incurred by through vehicles from those incurred by non-through vehicles. A reasonable approximation of the stop rate of through vehicles is the stop rate computed for the through lane group at the boundary intersection.

Stop rate is defined as the average number of full stops per vehicle. A full stop is defined to occur when a vehicle slows to zero (or a crawl speed, if in queue) in response to a change in signal indication from green to red. An arrival-departure polygon is shown in Exhibit 15-13 to illustrate the concept of full stop, as opposed to a partial stop. Each solid thin line in the exhibit that angles upward from left to right represents the trajectory of one vehicle. The time between trajectories represents the headway between vehicles (i.e., the inverse of flow rate \( q \)). The slope of the trajectory represents the vehicle’s speed. The curved portion of a trajectory indicates deceleration or acceleration. The horizontal portion of a trajectory indicates a stopped condition. The effective red \( r \) and effective green \( g \) times are dimensioned at the top of the exhibit.

Exhibit 15-13 shows the trajectories of eight vehicles. The first five trajectories (counting from left to right) have a horizontal component to their trajectory that indicates they have reached a full stop as a result of the red indication. The sixth trajectory has some deceleration and acceleration but the vehicle does not stop. This trajectory indicates a partial stop was incurred for the associated vehicle. The last two trajectories do not incur deceleration or acceleration, and the associated vehicles do not slow or stop. Thus, the number of full stops \( N_f \) is 5 and the number of partial stops \( N_p \) is 1. The total number of stops \( N_t \) is 6. The full stop rate is 0.63 stops/veh (= 5/8).

**Stop Rate Equation**

The following equation is used to estimate the stop rate for a through lane group.

\[
\begin{align*}
    h &= h_1 + 3600 \frac{Q_o}{vC} \\
    h_1 &= \frac{1 - P(1 + d_1/g)}{1 - PX} : \text{if } d_1 < (1 - P)gX \\
    h_1 &= \frac{(1 - P)(r - d_2)}{r - (1 - P)gX} : \text{if } d_1 > (1 - P)gX
\end{align*}
\]

\[
Q_o = 0.25c_aT \left[ X_a - \frac{Q_b}{c_aT} \right] + \left[ X_a - \frac{Q_b}{c_aT} \right]^2 + \frac{8k_bX_a}{c_aT} + \frac{16k_bQ_b}{(c_aT)^2}
\]

where,

- \( h \) = full stop rate, stops/veh;
- \( h_1 \) = deterministic stop rate, stops/veh;
- \( Q_o \) = overflow queue, veh;
- \( r \) = effective red time (= \( C - g \)), s;
- \( d_1 \) = deceleration-acceleration delay, s;
- \( k_b \) = second-term incremental factor; and
- \( Q_i \) = initial queue at the start of the analysis period, veh.
As noted in the previous section, Equation 15-20 tends to overestimate the overflow queue for an actuated phase when the average phase duration is used to estimate lane group capacity. To account for the ability of an actuated phase to prevent overflow, Equation 15-14 should be used to estimate the available capacity for all non-coordinated (or actuated) phases. For coordinated phases or pretimed phases, the available capacity is equal to the capacity \( c \). Additional guidance on this topic is provided with the discussion associated with Equation 15-14.

**Second-Term Incremental Factor**

The second-term incremental factor is calculated using the equations presented in this section. Equation 15-21 is used to estimate the factor value for coordinated or pretimed phases. Equation 15-22 is used to estimate the factor value for non-coordinated (or actuated) phases.

\[
k_B = 0.12 I \left( \frac{S g}{3600} \right)^{0.7} \quad \text{(coordinated phase)} \\
k_B = 0.10 I \left( \frac{S g}{3600} \right)^{0.6} \quad \text{(non-coordinated phase)}
\]

where,
- \( k_B \) = second-term incremental factor; and
- \( I \) = upstream filtering adjustment factor (from Exhibit 15-12).

For planning and preliminary design analyses, the segment through phase is a coordinated phase so Equation 15-21 should be used to estimate the factor value for the through lane group.

**Acceleration-Deceleration Delay**

The following equation is used to estimate the acceleration-deceleration delay associated with a specified threshold speed.

\[
d_a = \frac{[1.47 (S_a - S_s)]^2}{2 (1.47 S_s)} \left( \frac{1}{r_a} + \frac{1}{r_d} \right)
\]

where,
- \( d_a \) = deceleration-acceleration delay, s;
- \( S_s \) = threshold speed defining a stopped vehicle, mph;
- \( r_a \) = acceleration rate, ft/s^2;
- \( r_d \) = deceleration rate, ft/s^2;
- \( S_a \) = average speed (= 0.95 \( [25.6 + 0.47 \, S_p ] \)), mph; and
- \( S_p \) = speed limit, mph.

Practical considerations in the count of stopped vehicles require the specification of a non-zero threshold speed that can be used to identify when a vehicle is effectively stopped. Various definitions are used to describe this speed for the purpose of field measurement. These definitions typically allow a vehicle to be called “stopped” when it first reaches a crawl speed (say, 2 to 5 mph). A threshold speed of 5 mph is recommended for use with Equation 15-23.

The deceleration rate used in Equation 15-23 describes driver response to the red indication. It is an average value for all stopping vehicles. It is recognized that vehicles near the stop line at the onset of yellow may decelerate at twice the average rate. A value of 4.0 ft/s^2 is recommended for this application.

The acceleration rate describes driver speed change in response to the green indication. It is an average value for all starting vehicles. It is recognized that the first
vehicle in queue may accelerate at twice the average rate. A value of 3.5 ft/s² is recommended for this application.

Substitution of these recommended values in Equation 15-23 and reducing yields the following equation:

\[ d_a = 0.393 \frac{(S_a - 5.0)^2}{S_a} \]  \hspace{1cm} (15-24)

**DETERMINING TRAVEL SPEED**

Equation 15-25 is used to compute the average travel speed for the subject direction of travel along a segment.

\[ S_T = \frac{3600 L}{5280 (T_R + d)} \]  \hspace{1cm} (15-25)

where,
- \( S_T \) = average travel speed of through vehicles along the segment, mph;
- \( L \) = segment length, ft;
- \( T_R \) = segment running time, s; and
- \( d \) = control delay, s/veh.

The control delay used in Equation 15-25 is that incurred by the through lane group exiting the segment at the boundary intersection.

If two or more segments are being evaluated, Equation 15-26 is used to compute the overall average travel speed in the subject direction of travel.

\[ S_T = \frac{3600 \sum L}{5280 (\sum T_R + \sum d)} \]  \hspace{1cm} (15-26)

When using this equation, the length of each segment is added to obtain a total system length \( \sum L \). Similarly, the individual running times and delays of each segment are added to obtain a total travel time \( \sum (T_R + d) \).

**DETERMINING SPATIAL STOP RATE**

Equation 15-27 is used to compute the spatial stop rate for the subject direction of travel along a segment.

\[ H = \frac{5280 h + h_{other}}{L} \]  \hspace{1cm} (15-27)

where,
- \( H \) = spatial stop rate, stops/mi;
- \( h \) = full stop rate, stops/veh; and
- \( h_{other} \) = full stop rate due to other sources along the segment, stops/veh.

The full stop rate \( h \) used in Equation 15-27 is that incurred by the through lane group exiting the segment at the boundary intersection. In some situations, stops may be incurred at mid-segment locations due to pedestrian crosswalks, bus stops, or turns into unsignalized access points. The average stop rate associated with these other stops can be added to the numerator of Equation 15-27.

If two or more segments are being evaluated, Equation 15-28 is used to compute the overall spatial stop rate in the subject direction of travel.

\[ H = \frac{5280 (\sum h + \sum h_{other})}{\sum L} \]  \hspace{1cm} (15-28)
When using this equation, the length of each segment is added to obtain a total system length \( \Sigma L \). Similarly, the individual stop rates for each segment are added to obtain a total stop rate \( \Sigma (h + h_{other}) \).

**DETERMINING LEVEL OF SERVICE**

[This section may be deleted or re-written depending on the extent to which the HCQS committee incorporates the recommendations from NCHRP Project 3-70]

Exhibit 15-14 lists the level of service thresholds established for arterial streets and major collector streets. They are based on the average travel speed for through vehicles, expressed as a percentage of the base free-flow speed. Level of service can be determined for a segment or facility. However, it is more meaningful as an indicator of facility performance than segment performance. If demand volume exceeds capacity at any point on the facility, then average travel speed may not be a good indicator of level of service.

**EXHIBIT 15-14. Urban Street Level of Service.**

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Average Travel Speed as a Percentage of Base Free-Flow Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>85 % or more</td>
</tr>
<tr>
<td>B</td>
<td>67 to 84.9 %</td>
</tr>
<tr>
<td>C</td>
<td>50 to 66.9 %</td>
</tr>
<tr>
<td>D</td>
<td>40 to 49.9 %</td>
</tr>
<tr>
<td>E</td>
<td>30 to 39.9 %</td>
</tr>
<tr>
<td>F</td>
<td>less than 30 %</td>
</tr>
</tbody>
</table>

**III. APPLICATIONS**

This part of the chapter provides an overview of the urban street evaluation process. The first section describes the analysis levels that are supported by the methodology. The second section describes a procedure for defining the street segment or segments to be evaluated. The last section summarizes the computational steps included in the methodology. Worksheets are described in this section for use in planning and preliminary design analyses.

**LEVEL OF ANALYSIS**

The urban streets methodology can be used for planning, preliminary design, and operational analyses. A planning analysis is generally directed toward an evaluation of facility performance and is usually based on forecast traffic demands. The need to evaluate individual intersection traffic movements is relaxed and the focus is on overall indicators of through vehicle performance. Key inputs include traffic demand volume, segment length, and intersection lane allocations. Default values are used for many of the other input values.

Preliminary design analyses are primarily used to evaluate alternative design configurations for a new street segment or a segment that is planned for reconstruction. This analysis can be used to establish the basic number of through lanes needed, the need for turning lanes at intersections, or the feasibility of alternative locations for a mid-
segment access point. The input data used with this analysis are typically more accurately known than those used for a planning analysis and there are more inputs for which information is known. Less reliance is placed on the use of default values. Operational analyses are oriented toward the evaluation of an existing street segment or facility. A series of evaluations for a given segment (or facility) can be undertaken to develop improved signal timing plans or evaluate the benefit of changes in lane assignment. Most analyses are focused on quantifying the effects of specific, sometimes subtle, changes in the operation, geometry, or traffic demands on a segment. As a result, accuracy in the performance estimate of individual intersection movements is important. In this situation, inputs are based on field measurements and the use of default values is minimized to the extent possible.

SEGMENTING THE URBAN STREET

At the start of the analysis, the location and length of the urban street system to be considered must be defined. Based on consideration of the analysis scope, the analyst must determine whether it is appropriate to evaluate one segment, several segments, or the entire facility. The analyst should recognize that average travel speed is more meaningful when the performance of a facility is being evaluated.

The street segment is the basic unit of the analysis, as defined in the section titled “Urban Street System” in Chapter 10. In general, it represents a length of street bounded by two intersections. If the subject street is within a coordinated signal system, then the following rules apply when identifying the segment boundaries:

- A signalized intersection (or ramp terminal) is always used to define a segment boundary.
- Only intersections (or ramp terminals) at which the segment through movement is uncontrolled (e.g., a two-way stop controlled intersection) can exist along the segment between the boundaries.

If the subject street is not within a coordinated signal system, then the following rules apply when identifying the segment boundaries:

- An intersection (or ramp terminal) having a type of control that can impose on the segment through movement a legal requirement to stop or yield must always be used to define a segment boundary.
- An intersection (or ramp terminal) at which the segment through movement is uncontrolled (e.g., a two-way stop controlled intersection) may be used to define a segment boundary, but it is typically not done.

As discussed in Chapter 10, signalized segments that are very short (i.e., less than 400 ft) may experience spillback or demand starvation, both of which are not modeled by the urban streets methodology. If these conditions are present, then a different analysis tool should be considered. In contrast, a segment that is very long (i.e., more than 2 mi) is unlikely to be influenced by platoons formed by the upstream signal. In this case, the analyst should consider evaluating the segment as an uninterrupted flow highway segment with an isolated intersection.

COMPUTATIONAL STEPS

This section describes the urban streets methodology in terms of a sequence of computational steps. The steps are presented in the context of an evaluation of one street segment. If a facility is being evaluated, the steps are repeated for each segment and the travel speed is based on the total travel time for all segments that comprise the facility.

The sequence of computational steps varies based on whether the street system is coordinated or non-coordinated. The operational analysis of a coordinated system requires the most steps. The other combinations of analysis level and system control require a subset of these steps. These other combinations are described first because of their simplicity and because they illustrate key components of the methodology. The
The operational analysis of a coordinated system is discussed last. The focus of this discussion will be the additional computational steps required, relative to those required for the simpler combinations.

**Coordinated Segment - Planning Analysis**

The computational steps involved in a planning or preliminary design analysis of a coordinated street segment are described in this section. The analysis is focused on the performance of the segment through movement. The boundary intersection is signalized and the segment through movement is served by a coordinated phase.

The analysis consists of six computational steps. These steps are:

- Determine running time
- Determine proportion arriving during green
- Determine through control delay
- Determine through stop rate
- Determine travel speed
- Determine spatial stop rate

Each step is executed in the sequence presented in the preceding list. This sequence is illustrated by flow chart in Exhibit 15-15. The rectangles with rounded corners indicate the computational steps. The parallelograms indicate where input data are needed.

**EXHIBIT 15-15. Steps in Planning Analysis of Coordinated System.**

The computations associated with each step identified in Exhibit 15-15 are described in Part II - Methodology. These computations are conveniently illustrated in a series of worksheets, where each worksheet corresponds to one or more of the calculation steps.
The first of the computational worksheets is the Running Time worksheet. It is shown as Exhibit 15-16 (values shown apply to Example Problem 1, as discussed later). This worksheet combines input data describing the segment geometric design, speed limit, volume, and access point frequency to estimate the base free-flow speed. This speed is then adjusted for segment length effects to obtain the expected free-flow speed. The free-flow speed is then used to estimate a free-flow travel time which is adjusted for the proximity of other vehicles and any delay that is caused by turns into access points or other sources. Default values for the delay due to turns at mid-segment access points are listed in Exhibit 15-4 when other, more-accurate estimates of this delay are not available. The result of these adjustments is an estimate of the expected segment running time.

The second of the computational worksheets is the Proportion Arriving During Green worksheet. It is shown as Exhibit 15-17. This worksheet is designed for the analysis of the segment through lane group. It documents the calculation of the proportion of vehicles that arrive during the green indication. Input data include the effective green-to-cycle-length ratio and arrival type. As noted in a previous section, the methodology includes a more detailed procedure for computing the proportion arriving during green for the operational analysis of a coordinated system. Hence, this worksheet is only applicable to the planning or preliminary design analysis of a coordinated system.

The third computational worksheet is the Control Delay worksheet. It is shown as Exhibit 15-18. This worksheet is designed for the analysis of the segment through lane group. Input variables include the cycle length, effective green-to-cycle-length ratio, volume, saturation flow rate, initial queue, and lanes. The portion of arrivals during green is obtained from the previous worksheet.

The “available” effective green time is equal to the effective green time (i.e., the product of the cycle length and the input green-to-cycle-length ratio) because the lane group being analyzed is associated with a coordinated phase. As a result, the available capacity is equal to the capacity and the available-volume-to-capacity ratio is equal to the volume-to-capacity ratio.

The control delay is computed as the sum of three components. The first component to be computed is the uniform delay. The notation “\(\min(1, X)\)” is shown in the equation used to compute this delay. It means that the value to be substituted for this text is the smaller of 1.0 and the volume-to-capacity ratio.

The second delay component is the incremental delay. This delay is based on two factors. An incremental delay calibration factor of 0.5 is appropriate for the evaluation of a coordinated phase serving the segment through lane group. The upstream filtering adjustment factor requires the variable \(X_u\). This variable can be estimated as the volume-to-capacity ratio of the segment through lane group at the upstream signalized intersection. If the upstream intersection is one mile or more away, then the upstream filtering adjustment factor is equal to 1.0.

The third delay component is the initial queue delay. If the initial queue is zero, then this delay component also equals 0.0. When there is an initial queue of vehicles present at the start of the analysis (as observed at the beginning of the red indication), then the procedures in Appendix F of Chapter 16 are used to modify the calculation of the uniform delay component and to calculate the initial queue delay component.

The fourth computational worksheet is the Stop Rate worksheet. It is shown as Exhibit 15-19. This worksheet is designed for the analysis of the segment through lane group. The input variables are the same as those needed for the Control Delay worksheet with the addition of speed limit. Similarly, the calculation of available effective green and available capacity is the same. The average speed is estimated using the equation provided. If the average speed is known, it should be substituted for the estimated value.

The stop rate is computed as the sum of two components. The first component to be computed is the deterministic stop rate. Two equations are available for this computation. The correct equation to use is based on a check of the acceleration-deceleration delay relative to the computed threshold value.

### RUNNING TIME WORKSHEET

<table>
<thead>
<tr>
<th>General Information</th>
<th>Site Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analyst</td>
<td>JME</td>
</tr>
<tr>
<td>Street</td>
<td>Texas Avenue</td>
</tr>
<tr>
<td>Agency or Company</td>
<td>ACME Engr.</td>
</tr>
<tr>
<td>Jurisdiction</td>
<td></td>
</tr>
<tr>
<td>Date Performed</td>
<td>9/30/07</td>
</tr>
<tr>
<td>Analysis year</td>
<td>2007</td>
</tr>
<tr>
<td>Analysis Time Period</td>
<td>PM Peak</td>
</tr>
<tr>
<td>Level of Analysis</td>
<td>Planning</td>
</tr>
</tbody>
</table>

### Input Data

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EB/NB</td>
<td>WB/SB</td>
<td>EB/NB</td>
<td>WB/SB</td>
</tr>
</tbody>
</table>

#### Direction of travel

<table>
<thead>
<tr>
<th>EB/NB</th>
<th>WB/SB</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Segment Data

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Number of through lanes for length of segment (N), ln

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Speed limit (S<sub>a</sub>), mph

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Mid-segment volume (v), veh/h

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Total delay due to turns into access points (Σd<sub>ap</sub>), s/veh

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Delay due to other mid-segment sources (d<sub>other</sub>), s/veh

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Length of segment (L<sub>s</sub>), ft

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Width of upstream boundary intersection (w), ft

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Length of segment with restrictive median (L<sub>rm</sub>), ft

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Length of segment with non-restrictive median (L<sub>nr</sub>), ft

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Start-up lost time (t<sub>1</sub>), s

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Access Data

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Percent of street with curb on right-hand side (P<sub>curb</sub>), %

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Number of access points on right-hand side (N<sub>ap</sub>)

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Running Time Computation

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Adjusted segment length (L<sub>adj</sub>), ft

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Percent of segment length with restrictive median (P<sub>rm</sub>),%

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Speed constant (S<sub>o</sub>), mph

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Adjustment for cross section (f<sub>CS</sub>), mph

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Access point density (D<sub>s</sub>), access points/mi

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Adjustment for access points (f<sub>A</sub>), mph

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Base free-flow speed (S<sub>f0</sub>), mph

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Segment length adjustment factor (f<sub>L</sub>)

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Free-flow speed (S<sub>f</sub>), mph

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Proximity adjustment factor (f<sub>v</sub>)

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

##### Running time (T<sub>R</sub>), s

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: the first term in the running time equation is only applicable to segments with signalized or stop-controlled boundary intersections.
The second stop rate component is based on the overflow queue. This queue represents the average number of vehicles that are unserved at the end of the green interval. It is based on two factors. One factor is the upstream filtering adjustment factor. This value should be the same as that used in the Control Delay worksheet. The second factor is the second-term incremental factor. This factor includes the upstream filtering adjustment factor as one of its terms. The equation shown in the worksheet is appropriate for the analysis of a coordinated phase serving the segment through lane group. Equation 15-22 is used if the segment through lane group is a non-coordinated phase.

The fifth computational worksheet is the Travel Speed and Spatial Stop Rate worksheet. It is shown as Exhibit 15-20. This worksheet is designed for the analysis of the segment through lane group. The input values include segment length and the full stop rate associated with other mid-segment events (e.g., turns in access points). The other input data listed represent computed values and are obtained from the previous worksheets.

The worksheet is structured such that the travel speed and stop rate is computed for each segment and direction of travel. The last two columns are provided to compute the average travel speed and stop rate for the system. If the system consists of more than two segments, then additional worksheets are used and only the last two columns of one worksheet are used to record the system totals and averages.

In general, the last two columns in Exhibit 15-20 represent the sum of the segment values for the same direction of travel. The need for summation is indicated by an “s” symbol in the appropriate cells. The “a” symbol denotes the need for an average value. Specifically, this average is needed for the base free-flow speed for the system. It is computed as:

\[
S_f = \frac{\sum L}{\sum L/S_f}
\]  \hspace{1cm} (15-29)

The numerator in this equation represents the total length of the segments, and the denominator represents the sum of the travel time on each segment, when traveling at the base free-flow speed. The cells with an “e” symbol indicate that the equations in column 1 should be used. The values used in these equations are taken from the appropriate system total column.
### Exhibit 15-18. Control Delay Worksheet.

#### Control Delay Worksheet

<table>
<thead>
<tr>
<th>General Information</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Description</td>
<td>Texas Avenue, PM Peak</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Input Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis Period ((T)), h:</td>
<td>0.25</td>
</tr>
<tr>
<td>Direction of travel</td>
<td>EB/NB WB/SB EB/NB WB/SB</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Signal Timing Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cycle length ((C)), s</td>
<td>100 100 100 100</td>
</tr>
<tr>
<td>Effective green-to-cycle-length ratio ((g/C))</td>
<td>0.47 0.47 0.47 0.47</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Traffic Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane group volume ((v)), veh/h</td>
<td>968 950 950 968</td>
</tr>
<tr>
<td>Lane group saturation flow rate ((s)), veh/h/ln</td>
<td>1800 1800 1800 1800</td>
</tr>
<tr>
<td>Initial queue ((Q_b)), veh</td>
<td>0 0 0 0</td>
</tr>
<tr>
<td>Proportion of arrivals during green ((P))</td>
<td>0.67 0.31 0.66 0.34</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Geometric Design Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane group lanes ((N)), ln</td>
<td>2 2 2 2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Delay Computation</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Available effective green time ((g_a)), s</td>
<td>Equ. 15-15 or 15-16 47 47 47 47</td>
</tr>
<tr>
<td>Capacity ((c)), veh/h</td>
<td>(c = N s g/C) 1692 1692 1692 1692</td>
</tr>
<tr>
<td>Available capacity ((c_a)), veh/h</td>
<td>(c = N s g_g/C) 1692 1692 1692 1692</td>
</tr>
<tr>
<td>Volume-to-capacity ratio ((X))</td>
<td>(X = v/c) 0.57 0.56 0.56 0.57</td>
</tr>
<tr>
<td>Available volume-to-capacity ratio ((X_a))</td>
<td>(X_a = v/c_a) 0.57 0.56 0.56 0.57</td>
</tr>
<tr>
<td>Uniform delay ((d_1)), s/veh</td>
<td>(d_1 = \frac{0.5 C (1-g/C)^2}{1 - m \ln(1, X) g/C} ) 19.2 19.1 19.1 19.2</td>
</tr>
<tr>
<td>Incremental delay calibration factor ((k))</td>
<td>(Exhibit 16-3) 0.50 0.50 0.50 0.50</td>
</tr>
<tr>
<td>Upstream filtering adjustment factor ((I))</td>
<td>(I = 1.0 - 0.91 X_a^{0.86} ) 0.80 0.80 0.80 0.80</td>
</tr>
<tr>
<td>Incremental delay ((d_2)), s/veh</td>
<td>(d_2 = 900 T \left{ \left( X_a - 1 \right) + \sqrt{\left( X_a - 1 \right)^2 + \frac{8 k \ln(1, X)}{c_a T}} \right} ) 1.13 1.08 1.08 1.13</td>
</tr>
<tr>
<td>Initial queue delay ((d_3)), s/veh</td>
<td>(Chapter 16, Appendix F) 0 0 0 0</td>
</tr>
<tr>
<td>Flow ratio ((y))</td>
<td>(y = m h(1, X) g/C) 0.27 0.26 0.26 0.27</td>
</tr>
<tr>
<td>Progression adjustment factor ((PF))</td>
<td>(PF = \frac{1 - P}{(1 - g/C)^2} \left( 1 - y \right) \left( \frac{1 - g/C + y (1 - P C g)}{1 - y P C g} \right) ) 0.57 1.35 0.60 1.28</td>
</tr>
<tr>
<td>Control delay ((d)), s/veh</td>
<td>(d = d_1 (PF) + d_2 + d_3) 12.1 26.8 12.6 25.7</td>
</tr>
</tbody>
</table>
### Exhibit 15-19. Stop Rate Worksheet.

**Stop Rate Worksheet**

<table>
<thead>
<tr>
<th>General Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Description</td>
</tr>
</tbody>
</table>

#### Input Data

<table>
<thead>
<tr>
<th>Analysis Period (T), h:</th>
<th>0.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction of travel</td>
<td>EB/NB</td>
</tr>
</tbody>
</table>

#### Signal Timing Data

<table>
<thead>
<tr>
<th>Cycle length (C), s</th>
<th>100</th>
<th>100</th>
<th>100</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective green-to-cycle-length ratio (g/C)</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
<td>0.47</td>
</tr>
</tbody>
</table>

#### Traffic Data

<table>
<thead>
<tr>
<th>Lane group volume (v), veh/h</th>
<th>968</th>
<th>950</th>
<th>950</th>
<th>968</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane group saturation flow rate (s), veh/h/ln</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
</tr>
<tr>
<td>Initial queue (Qb), veh</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Proportion of arrivals during green (P)</td>
<td>0.67</td>
<td>0.31</td>
<td>0.66</td>
<td>0.34</td>
</tr>
<tr>
<td>Speed limit (Spl), mph</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
</tr>
</tbody>
</table>

#### Geometric Design Data

| Lane group lanes (N), ln | 2 | 2 | 2 | 2 |

#### Stop Rate Computation

<table>
<thead>
<tr>
<th>Available effective green time (ga), s</th>
<th>47</th>
<th>47</th>
<th>47</th>
<th>47</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective green time (g), s</td>
<td>g = C (g/C)</td>
<td>47</td>
<td>47</td>
<td>47</td>
</tr>
<tr>
<td>Effective red time (r), s</td>
<td>r = C - g</td>
<td>53</td>
<td>53</td>
<td>53</td>
</tr>
<tr>
<td>Capacity (c), veh/h</td>
<td>c = Nsg/C</td>
<td>1692</td>
<td>1692</td>
<td>1692</td>
</tr>
<tr>
<td>Available capacity (ca), veh/h</td>
<td>c = Nsg/C</td>
<td>1692</td>
<td>1692</td>
<td>1692</td>
</tr>
<tr>
<td>Volume-to-capacity ratio (X)</td>
<td>X = v/c</td>
<td>0.57</td>
<td>0.56</td>
<td>0.56</td>
</tr>
<tr>
<td>Available volume-to-capacity ratio (Xa)</td>
<td>Xa = v/ca</td>
<td>0.57</td>
<td>0.56</td>
<td>0.56</td>
</tr>
<tr>
<td>Average speed (Sa), mph</td>
<td>Sa</td>
<td>39.9</td>
<td>39.9</td>
<td>39.9</td>
</tr>
<tr>
<td>Threshold accel.-decel. delay, s</td>
<td>(1 - P)gX</td>
<td>8.8</td>
<td>18.1</td>
<td>9.0</td>
</tr>
<tr>
<td>Acceleration-deceleration delay (ds)</td>
<td>ds</td>
<td>12.0</td>
<td>12.0</td>
<td>12.0</td>
</tr>
<tr>
<td>Deterministic stop rate (h1), stops/veh</td>
<td>h1 = 1 - Pd / (1 - PX) : If ds &lt; (1 - P)gX</td>
<td>0.30</td>
<td>0.74</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td>h1 = (1 - P)(r - ds) / r(1 - P)gX : If ds &gt; (1 - P)gX</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upstream filtering adjustment factor (f)</td>
<td>f = 1.0 - 0.91Xa2</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>Second-term incremental factor (k8)</td>
<td>k8 = 0.12</td>
<td>0.87</td>
<td>0.87</td>
<td>0.87</td>
</tr>
<tr>
<td>Overflow queue (Qo), veh</td>
<td></td>
<td>1.15</td>
<td>1.10</td>
<td>1.10</td>
</tr>
<tr>
<td>Full stop rate (h), stops/veh</td>
<td>h = h1 + 3600Qo / VC</td>
<td>0.35</td>
<td>0.78</td>
<td>0.36</td>
</tr>
</tbody>
</table>
EXHIBIT 15-20. Travel Speed and Spatial Stop Rate Worksheet.

<table>
<thead>
<tr>
<th>General Information</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>System Total</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Project Description</strong></td>
<td>Texas Avenue, PM Peak</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Input Data</strong></th>
<th>1</th>
<th>2</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Direction of travel</strong></td>
<td>EB/NB</td>
<td>WB/SB</td>
<td>EB/NB</td>
<td>WB/SB</td>
</tr>
<tr>
<td><strong>Length of segment (L), ft</strong></td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
</tr>
<tr>
<td><strong>Base free-flow speed (Sfo), mph</strong></td>
<td>40.8</td>
<td>40.8</td>
<td>40.8</td>
<td>40.8</td>
</tr>
<tr>
<td><strong>Running time (TR), s</strong></td>
<td>33.7</td>
<td>33.7</td>
<td>33.7</td>
<td>33.7</td>
</tr>
<tr>
<td><strong>Control delay (d), s/veh</strong></td>
<td>12.1</td>
<td>26.8</td>
<td>12.6</td>
<td>25.7</td>
</tr>
<tr>
<td><strong>Full stop rate (h), stops/veh</strong></td>
<td>0.35</td>
<td>0.78</td>
<td>0.36</td>
<td>0.75</td>
</tr>
<tr>
<td><strong>Full stop rate due to other mid-segment sources (hother), stops/veh</strong></td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

**Travel Speed Computation**

| **Travel time (TR), s** | 45.8 | 60.6 | 46.3 | 59.4 | e 92.2 | e 120 |
| **Travel speed (ST), mph** | 26.8 | 20.3 | 26.5 | 20.7 | e 26.6 | e 20.5 |

**Spatial Stop Rate Computation**

| **Total stop rate (hT), stops/veh** | 0.35 | 0.78 | 0.36 | 0.75 | e 0.71 | e 1.53 |
| **Spatial stop rate (H), stops/mi** | 1.01 | 2.28 | 1.06 | 2.20 | e 1.04 | e 2.24 |

**Level of Service Computation**

| **Travel speed as a percentage of base free-flow speed** | 65.7 | 49.7 | 65.0 | 50.7 | e 65.3 | e 50.2 |

| **Level of service (Exhibit 15-14)** | C | D | C | C | C | C |

Note: 1 - Calculation code: s - segment sum; a - segment average (see text); e - use equation in column 1.

---

Non-Coordinated Segment Analysis

The computational steps involved in the planning, preliminary design, or operational analysis of a non-coordinated street segment are described in this section. The analysis is focused on the performance of the segment through movement. The boundary intersections are signalized or unsignalized. If an intersection is signalized, the segment through movement is served by a non-coordinated phase.

The analysis consists of three computational steps. These steps are:

- Determine running time
- Determine through control delay
- Determine travel speed

This sequence is illustrated by flow chart in Exhibits 15-21 and 15-22. Exhibit 15-21 is applicable when the boundary intersections are signalized. Exhibit 15-22 is applicable when the boundary intersections are unsignalized (i.e., all-way stop controlled or roundabout). The rectangles with rounded corners indicate the computational steps. The parallelograms indicate where input data are needed.
The computations associated with each step identified in Exhibits 15-21 and 15-22 are described in Part II - Methodology. These computations are conveniently illustrated in a series of worksheets, where each worksheet corresponds to one or more of the calculation steps.

The first of the computational worksheets is the Running Time worksheet. It was shown previously as Exhibit 15-16. The delay due to turns at mid-segment access points.
is estimated. Default values for this delay are listed in Exhibit 15-4 when other, more-accurate estimates of this delay are not available. All calculations using this worksheet are as described in the section titled Coordinated Segment - Planning Analysis.

As indicated in both Exhibits 15-21 and 15-22, the second step involves the computation of control delay at the boundary intersection. If the boundary intersection is signalized, then the Control Delay worksheet in Exhibit 15-18 is used. If the phase serving the segment through movement is pretimed, then the effective-green-to-cycle length ratio is an input value. If the phase is actuated, then an average value for this ratio is computed using the procedure described in Appendix B of Chapter 16.

Arrivals to the intersection during the red and green intervals are effectively random when averaged over the analysis period. Thus, the proportion of arrivals during green $P$ is estimated as being equal to the effective green-to-cycle-length ratio.

If the phase serving the segment through lane group is pretimed, then Equation 15-16 is used to estimate the available effective green time. If the phase is actuated, then Equation 15-15 is used. All other calculations using this worksheet are as described in the section titled Coordinated Segment - Planning Analysis.

If the boundary intersection is unsignalized, then the control delay for the segment through lane group is computed using the methodology described in the appropriate chapter. Specifically, if the intersection is all-way stop controlled, then the procedure in Chapter 17 is used to estimate the control delay. If the intersection is a roundabout, then the procedure described in a forthcoming chapter is used.

The computational steps for non-coordinated systems do not include the calculation of stop rate because stop rate is not highly relevant to the performance of these systems.

The last step involves the Travel Speed and Spatial Stop Rate worksheet. It is shown as Exhibit 15-20. This worksheet is designed for the analysis of the segment through lane group. The spatial stop rate computation portion can be used if procedures are available from other chapters for estimating through movement stop rate. The other input data listed represent computed values and are obtained from the previous worksheets. All other calculations using this worksheet are as described in the section titled Coordinated Segment - Planning Analysis.

**Coordinated Segment - Operational Analysis**

The computational steps involved in an operational analysis of a coordinated street segment are described in this section. The analysis is based on consideration of all traffic movements at the boundary intersection or at a mid-segment access point. It is also based on the concurrent evaluation of both directions of travel such that key traffic and signal timing interactions are considered. The boundary intersection is signalized and the segment through movement is served by a coordinated phase.

The analysis consists of eight computational steps. These steps are:

- Determine traffic demand adjustments
- Determine running time
- Determine proportion arriving during green
- Determine signal phase duration
- Determine through control delay
- Determine through stop rate
- Determine travel speed
- Determine spatial stop rate

Each step is executed in the sequence presented in the preceding list. This sequence is illustrated by flow chart in Exhibit 15-23. The rectangles with rounded corners indicate the computational steps. The parallelograms indicate where input data are needed. The diamond shapes indicate decision points.

Input volume, geometry, signal settings

Demand Adjustment Module
- Entry demand > entry capacity?
  - Yes: Constrain upstream segment entry flows
  - No: Balance entry and exit flows between junctions
- Entry volume = exit volume?
  - Yes: Compute origin-destination matrix
  - No: Sustained spillback?
    - Yes: Report time of spillback
    - No: Compute discharge flow profile at upstream intersection

Segment Analysis Module
- Compute segment running time
- Compute arrival flow profile at downstream junctions
- Compute conflicting flow rate of platoons passing access points

Signalized Intersection Module
- Compute call rate to extend a phase and to activate a phase
- Compute queue accumulation polygon
  - Permissive service time
  - Lane volume distribution
  - Lane saturation flow rate
  - Uniform delay
  - Stop rate of platoon arrivals
- Compute lane volume distribution
- Compute maximum allowable headway to extend green
- Compute equivalent maximum green based on force offs
- Compute average phase duration

Delay Due to Turns Module
- Compute delays due to left and right turns
- Report delays due to turns at each access point

Performance Measures Module
- Phase duration convergence with previous iteration?
  - Yes: Compute incremental delay and overflow stop rate
  - No: Compute travel speed and spatial stop rate for segment
- Report segment delay, travel time, stop rate, and travel speed
The computations associated with each step identified in Exhibit 15-23 are described in Part II - Methodology. Several of these computations have been discussed in the two previous sections. However, as the flow chart indicates, this analysis is much more detailed in terms of the factors considered. A consequence of this added detail is that the calculations are iterative and require implementation in software for efficient implementation. Hence, there are no worksheets for this analysis level.

The focus of this section is an overview discussion of the computational steps. A more detailed discussion of the methodology is described in Appendix A.

The first of the computational steps is the traffic demand adjustment. There are four calculations undertaken in this step. The first calculation checks the volume of the three traffic movements entering the segment at the upstream intersection. The check is used to determine if a movement has its discharge volume metered by its phase capacity. If the volume exceeds capacity for a movement, then the discharge volume is reduced such that it equals the capacity.

The second calculation checks the balance between entry and exit volumes at each intersection and access point. This check is used to verify whether the volume entering the segment at the upstream intersection arrives at each downstream junction. If there is a mis-match in this volume, then the segment left-turn, through, and right-turn movements at the downstream junction are adjusted such that their sum equals the upstream entry volume.

The third calculation is the distribution of each upstream entry volume to each of the downstream exit movements. This distribution represents the origin-destination matrix. The algorithm used to calculate this matrix is described in the section titled, Intersection Turning Movements in Chapter 10.

The last calculation in this step checks the queue length on the segment extending back from the downstream signalized intersection. If a downstream movement exceeds its capacity, then the time that spillback occurs is computed and reported. If this time of occurrence is shorter than the analysis period, then the methodology may not yield accurate performance estimates. In this situation, the analyst should consider reducing the analysis period such that it ends before spillback occurs or consider the use of an analysis tool that more accurately models spillback events.

The second computational step is focused on the calculation of running time. However, it begins with the calculation of the discharge flow profile of vehicles entering the segment. This profile describes the flow rate of entry vehicles at each 1-s time step in the signal cycle, as influenced by the upstream signal timing. The second calculation produces the average running time and is based on the calculations described in Exhibit 15-16.

The third computational step combines the running time with a platoon dispersion algorithm to calculate the arrival flow profile at each downstream access point or intersection. This profile describes the flow rate of arriving vehicles at each 1-s time step in the signal cycle. It is used to calculate the proportion of vehicles arriving during the green indication. It is also used to calculate the proportion of time blocked by a passing platoon at each access point. This latter value is an input to the two-way stop controlled intersection evaluation methodology described in Chapter 17.

The fourth computational step is focused on the determination of average phase duration. There are five calculations associated with this step. The first calculation determines the rate at which the vehicles place a “call” for service to the controller. Higher rates are associated with higher volumes and tend to extend the phase longer.

The second calculation defines the queue accumulation polygon associated with each intersection lane group. If shared lanes exist in a lane group, then a polygon is constructed for each lane. The primary purpose of this polygon is to determine the queue service time. However, it is also used in a subsequent step to estimate the deterministic delay and stop rate for each lane group.
The third calculation quantifies the maximum allowable headway associated with each non-coordinated signal phase. It represents the flow-weighted average of the maximum allowable headway associated with each lane group served by the phase.

The fourth calculation yields the equivalent maximum green period associated with the non-coordinated phases. This maximum is computed using the input force-off mode and phase splits.

The fifth calculation determines the average phase duration using Equation 15-8. One element of this calculation is the green extension time. It is computed using the aforementioned call rate and maximum allowable headway. The calculation procedure is derived to ensure that the green interval does not exceed the equivalent maximum green.

The fifth computational step is focused on the calculation of delay to segment through vehicles due to turns at access points. This step consists of three calculations. The first calculation quantifies the volume in each through lane that is approaching the access point. This calculation is based on an algorithm that distributes volume to the approach lanes based on the anticipated delays in the inside and outside lanes. The second calculation yields the delay due to vehicles turning right from the segment into an access point. The third calculation yields the delay due to vehicles turning left from the segment. The algorithm associated with this calculation considers the presence of a left-turn bay (if any) in the calculation of delay. It also considers the proportion of time that the left-turn movement is blocked by a platoon in the opposing traffic stream.

At the conclusion of the fifth computational step, each step is repeated using the information from the first sequence of calculations to update the assumed values and improve the accuracy of the second solution. The process is repeated several times and converges to a solution within 10 to 15 iterations.

The last computational step computes the performance measures for the segment. It consists of two calculations. The first calculation yields the incremental delay term and the overflow stop rate for each lane group. The second calculation uses the delay and stop rate to estimate the travel speed and spatial stop rate, respectively. As noted previously, the deterministic delay and stop rate were computed in conjunction with the queue accumulation polygon.

### IV. EXAMPLE PROBLEMS

<table>
<thead>
<tr>
<th>Problem No.</th>
<th>Description</th>
<th>Level of Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Coordinated system</td>
<td>Planning</td>
</tr>
<tr>
<td>2</td>
<td>Coordinated system</td>
<td>Operational</td>
</tr>
</tbody>
</table>
EXAMPLE PROBLEM 1

The Urban Street. The total length of an undivided urban street is 3600 ft. It consists of two segments and three signalized boundary intersections. The street has a four-lane cross section with two lanes in each direction. There are left-turn bays on the major street at each signalized intersection. Each segment has four driveways on each side of the street; however, only two driveways are considered to be active. The two active driveways for each segment are shown in the schematic below.

The Question. What is the travel speed, spatial stop rate, and level of service by segment and for the total length of the street in both directions of travel?

The Facts.
- Speed limit = 35 mph
- Boundary intersection width = 50 ft
- Signal control: coordinated actuated
- Cycle length = 100 s
- Initial queue = 0 veh
- Percent left turns = 6%
- Percent right turns = 8%
- Mid-segment volume = 1150 veh/h
- Percent of street with curb = 70%
- g/C ratio = 0.47
- Analysis period = 0.25 h
- Through lane group lanes = 2

Outline of Solution. Default values are: saturation flow rate = 1800 veh/h/ln; start-up lost time = 2.0 s. The following steps describe computations for the two segments.

Selected Calculations.

<table>
<thead>
<tr>
<th>Step</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Compute the total delay due to turns into access points</td>
</tr>
<tr>
<td></td>
<td>Mid-segment lanes = 2 lanes</td>
</tr>
<tr>
<td></td>
<td>Mid-segment lane volume = 575 veh/h/ln</td>
</tr>
<tr>
<td></td>
<td>Interpolation in Exhibit 15-4 to obtain 0.37 s/veh/pt through vehicle delay.</td>
</tr>
<tr>
<td></td>
<td>Number of active access points = 2</td>
</tr>
<tr>
<td></td>
<td>Percent turns = 7% [= (6 + 8)/2]</td>
</tr>
<tr>
<td></td>
<td>Total delay per access pt. = 7/10 x 0.37</td>
</tr>
<tr>
<td></td>
<td>= 0.26 s/veh/pt</td>
</tr>
<tr>
<td></td>
<td>Total delay per segment = 2 x 0.26</td>
</tr>
<tr>
<td></td>
<td>= 0.52 s/veh</td>
</tr>
<tr>
<td>2.</td>
<td>Compute upstream filtering factor</td>
</tr>
<tr>
<td></td>
<td>Upstream X ratio for eastbound intersection 2 delay calculation is not available (i.e., no data were provided for intersection 1). This ratio is estimated to equal the X ratio for intersection 2.</td>
</tr>
<tr>
<td></td>
<td>A similar assumption is made for the upstream X ratio for the westbound intersection 2</td>
</tr>
</tbody>
</table>

Results. The calculations are shown in Exhibits 15-16 to 15-20. The travel speed for the eastbound direction is 26.6 mph and for the westbound direction is 20.5 mph. The eastbound and westbound spatial stop rate is 1.04 and 2.24 stops.mi, respectively. The travel speed for the eastbound direction is 65 percent (=26.6/40.8 x 100) of the base free-flow speed and, according to Exhibit 15-14, corresponds to level of service “C.” The westbound level of service is similarly computed to be “C.”
EXAMPLE PROBLEM 2

The Urban Street. The total length of an undivided urban street is 3600 ft. It consists of two segments and three signalized boundary intersections. The street has a four-lane cross section with two lanes in each direction. There are left-turn bays on the major street at each signalized intersection. Each segment has four driveways on each side of the street; however, only two driveways are considered to be active. The two active driveways for each segment are shown in the schematic below (i.e., D1, D2, D3, and D4).

The turn movement volume for each intersection and driveway on segment 1 is shown below. The volume on segment 2 is the same (i.e., all intersections have the same volume and all driveways have the same volume).

The Question. What is the travel speed, spatial stop rate, and level of service by segment and for the total length of the street in both directions of travel?

The Facts.
- Speed limit = 35 mph
- Boundary intersection width = 50 ft
- Signal control: coordinated actuated
- Analysis period = 0.25 h
- One-lane left-turn bay at each signal
- Protected left-turn mode on major street
- Leading left-turn phases on major street
- Offsets referenced to end of green
- Through phase change period = 4.0 s
- Through phase splits = 25 s
- Through lane group lanes = 2
- Percent of street with curb = 70%
- Cycle length = 100 s
- Initial queue = 0 veh
- One-lane right-turn bay on major street at each signal
- Protected-permissive left-turn mode on minor street
- Offsets of 0, 30, and 60 s for signals 1, 2, and 3, respectively
- Fixed force mode
- Left-turn phase change period = 3.0 s
- Left-turn phase splits = 20 s

Outline of Solution. Default values are: saturation flow rate = 1800 veh/h/ln; start-up lost time = 2.0 s; stop line detection zone length = 40 ft; passage time = 2.0 s; minimum green = 5 s; arrival type 3 for all external movements; 35 mph speed limit for all external movements; dual entry for through phases; simultaneous gap-out for all phases, no recall to non-coordinated phases, turn bay length = 200 ft. The calculation of free-flow speed is the same as computed for Example Problem 1; however, all calculations for an operations analysis of a coordinated system are automated using software. Selected output from these calculations is described in the following paragraphs.
Exhibit 15-24 summarizes the analysis of the individual traffic movements at intersection 1. A similar summary is obtained for intersections 2 and 3. The movement numbers for the left-turn and through movements follow the NEMA phase numbering convention. The right-turn movement number is equal to the through movement number plus 10.

**EXHIBIT 15-24. Intersection 1 Movement-Based Output.**

<table>
<thead>
<tr>
<th>Movement</th>
<th>EB</th>
<th>EB</th>
<th>EB</th>
<th>WB</th>
<th>WB</th>
<th>WB</th>
<th>NB</th>
<th>NB</th>
<th>NB</th>
<th>SB</th>
<th>SB</th>
<th>SB</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
</tr>
<tr>
<td>Volume, veh/h</td>
<td>200.00</td>
<td>1000.00</td>
<td>10.00</td>
<td>189.86</td>
<td>949.29</td>
<td>9.49</td>
<td>100.00</td>
<td>500.00</td>
<td>50.00</td>
<td>100.00</td>
<td>500.00</td>
<td>50.00</td>
</tr>
<tr>
<td>Lanes</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>Bay Length, ft</td>
<td>200</td>
<td>0</td>
<td>200</td>
<td>200</td>
<td>0</td>
<td>200</td>
<td>200</td>
<td>0</td>
<td>0</td>
<td>200</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Saturation Flow Rate, veh/h/in</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
</tr>
<tr>
<td>Arrival Type</td>
<td>3.0</td>
<td>4.0</td>
<td>3.0</td>
<td>3.0</td>
<td>4.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Initial Queue</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Speed Limit, mph</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
</tr>
<tr>
<td>Detector Length, ft</td>
<td>40</td>
<td>0</td>
<td>40</td>
<td>40</td>
<td>0</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>0</td>
<td>40</td>
<td>40</td>
<td>0</td>
</tr>
<tr>
<td>Capacity, veh/h</td>
<td>233.68</td>
<td>1701.70</td>
<td>723.22</td>
<td>222.37</td>
<td>1677.91</td>
<td>713.11</td>
<td>219.88</td>
<td>626.21</td>
<td>62.62</td>
<td>219.88</td>
<td>626.21</td>
<td>62.62</td>
</tr>
<tr>
<td>Discharge Volume, veh/h</td>
<td>0.00</td>
<td>1000.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>50.00</td>
<td>50.00</td>
<td>50.00</td>
<td>50.00</td>
<td>50.00</td>
</tr>
<tr>
<td>Proportion Arriving On Green</td>
<td>0.137</td>
<td>0.630</td>
<td>0.473</td>
<td>0.160</td>
<td>0.321</td>
<td>0.160</td>
<td>0.063</td>
<td>0.194</td>
<td>0.194</td>
<td>0.063</td>
<td>0.194</td>
<td>0.194</td>
</tr>
<tr>
<td>Approach Volume, veh/h</td>
<td>1210.00</td>
<td>1148.64</td>
<td>650.00</td>
<td>650.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Approach Delay, s/veh</td>
<td>18.908</td>
<td>29.028</td>
<td>37.426</td>
<td>37.426</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Approach Stop Rate, stops/veh</td>
<td>0.488</td>
<td>0.781</td>
<td>0.837</td>
<td>0.837</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The volumes shown in Exhibit 15-24 for the eastbound (EB), northbound (NB), and southbound (SB) movements are identical to the input volumes. The westbound (WB) volumes were reduced from the input volumes by the Volume Balance routine. This reduction occurred because the westbound volume input for this intersection exceeded the volume departing the upstream driveway (i.e., driveway 1).

The bay length shown is used only in the calculation of spillback time. Spillback occurrence is discussed in a subsequent paragraph.

Arrival type is shown for each left-turn (LT), through (TH), and right-turn (RT) movement; however, the value shown for each internal movement (i.e., the westbound movements at this intersection) is only used for the first iteration of calculations. Improved estimates of progression quality are obtained with each iteration and the input value of arrival time for internal movements is ignored during subsequent iterations.

The initial queue is used in the calculation of spillback time and in the calculation of stop rate. It is not used to calculate delay (i.e., the procedure in Appendix F of Chapter 16 is not implemented in this methodology).

Discharge volume is computed by the Capacity Constraint routine for those movements that enter a segment. At signalized intersection 1, the movements entering segment 1 are the eastbound through movement, northbound right-turn movement, and southbound left-turn movement. A value of 0.0 veh/h is shown for all other movements and indicates that they are not relevant to this calculation. If volume exceeds capacity for any given movement, then the discharge volume is set equal to the capacity. Otherwise, the discharge volume is equal to the movement volume.

The proportion arriving during green $P$ is computed for internal movements by the routine of the same name. It is computed from the input arrival type for external movements.

The last three rows in Exhibit 15-24 represent summary statistics for the approach. The approach volume represents the sum of the three movement volumes. The delay and stop rate statistics are computed as volume-weighted averages for the lane groups served on the approach.

Exhibit 15-25 summarizes the output for Intersection 1 using a controller perspective. The controller has eight timing functions (or timers), with functions 1 to 4 representing ring 1 and functions 5 to 8 representing ring 2. When a lead/lead left-turn sequence is used, then there is a one-to-one match between the timing function number and the phase number.

The timing function construct is essential to the modeling of a ring-based signal controller. Timers always occur in the same numeric sequence (i.e, 1 then 2 then 3 then 4 in ring 1; 5 then 6 then 7 then 8 in ring 2). The practice of associating movements to phases (i.e., major-street through movement to phase 2) coupled with the need for lagging left-turn phases and split phasing, creates the situation where phases do not always time in sequence. For example, with a lagging left-turn phase sequence, Phase 2 times first and then Phase 1 times. The modern controller accommodates the assignment of phases to timing functions by allowing the ring structure to be redefined by direct user input or by time-of-day settings. Specification of this structure is automated in the methodology by the predetermined assignment of phases to timers. The methodology is based on modeling timer input, controller response, and timer output--where timer parameters are based on phase and movement inputs.
EXHIBIT 15-25. Intersection 1 Timer-Based Phase Output.

<table>
<thead>
<tr>
<th>Timer Data</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assigned Phase</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>Case No.</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>4</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Phase Duration (G+Y+Rc), s</td>
<td>16.00</td>
<td>51.27</td>
<td>9.28</td>
<td>23.44</td>
<td>16.67</td>
<td>50.61</td>
<td>9.28</td>
<td>23.44</td>
</tr>
<tr>
<td>Change Period (Y+Rc), s</td>
<td>3.00</td>
<td>4.00</td>
<td>3.00</td>
<td>4.00</td>
<td>3.00</td>
<td>4.00</td>
<td>3.00</td>
<td>4.00</td>
</tr>
<tr>
<td>Phase Start Time, s</td>
<td>36.73</td>
<td>52.73</td>
<td>4.00</td>
<td>13.28</td>
<td>36.73</td>
<td>53.39</td>
<td>4.00</td>
<td>13.28</td>
</tr>
<tr>
<td>Phase End Time, s</td>
<td>52.73</td>
<td>4.00</td>
<td>13.28</td>
<td>36.73</td>
<td>53.39</td>
<td>4.00</td>
<td>13.28</td>
<td>36.73</td>
</tr>
<tr>
<td>Maximum Allowable Headway (MAH), s</td>
<td>2.97</td>
<td>0.00</td>
<td>2.97</td>
<td>0.00</td>
<td>2.97</td>
<td>0.00</td>
<td>2.97</td>
<td>3.00</td>
</tr>
<tr>
<td>Equivalent Max. Green, s</td>
<td>29.27</td>
<td>0.00</td>
<td>17.00</td>
<td>31.72</td>
<td>29.27</td>
<td>0.00</td>
<td>17.00</td>
<td>31.72</td>
</tr>
<tr>
<td>Queue Clearance Time, s</td>
<td>12.80</td>
<td>0.00</td>
<td>6.61</td>
<td>16.80</td>
<td>13.43</td>
<td>0.00</td>
<td>6.61</td>
<td>16.80</td>
</tr>
<tr>
<td>Green Extension Time, s</td>
<td>0.269</td>
<td>0.00</td>
<td>2.638</td>
<td>0.00</td>
<td>0.263</td>
<td>0.00</td>
<td>2.638</td>
<td>0.00</td>
</tr>
<tr>
<td>Probability of Phase Call</td>
<td>0.995</td>
<td>0.00</td>
<td>0.938</td>
<td>0.00</td>
<td>0.996</td>
<td>0.00</td>
<td>0.938</td>
<td>1.00</td>
</tr>
<tr>
<td>Probability of Max Out</td>
<td>0.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.012</td>
</tr>
</tbody>
</table>

The signalized intersections in this example problem all have lead/lead left-turn sequences. Hence, the timer number is equal to the phase number (e.g., the westbound movement is associated with phase 1, which is assigned to timer 1).

The case number shown in Exhibit 15-25 describes the combination of left-turn mode and use of shared or exclusive lanes.

The phase duration shown in represents the average phase duration estimated by the average phase duration routine. It includes the green, yellow change, and red clearance intervals. For phase 2, the average green interval duration can be computed as 47.27 s (= 51.27 - 4.00).

The phase start time represents the time the timer (and phase) starts, relative to system time 0.0. For phase 2, the start time is 52.73 s. The end of the green interval associated with this phase is 100.0 s (= 52.73 + 47.27). This time is equal to the cycle length, so the end of green actually occurs at 0.0 s. This result is expected because phase 2 is the coordinated phase and the offset to the end of phase 2 (relative to system time 0.0) was input as 0.0 s.

The phase end time represents the time the timer (and phase) ends, relative to system time 0.0. For phase 2, the end of the green interval occurs at 0.0 s and the end of the phase occurs 4.0 s later (i.e., the change period duration).

The equivalent maximum allowable headway is an output from the routine of the same name. The same can be said for the equivalent maximum green. The queue clearance time is an output of the Queue Accumulation Polygon routine. This time equals the queue service time plus the start-up lost time. These three statistics are shown only for the non-coordinated phases. They are not relevant to the calculation of coordinated phase duration or delay.

The green extension time, the probability of phase call, and the probability of max-out are outputs from the Average Phase Duration routine. The green extension time represents the time the green interval is extended by arriving vehicles. The probability of a phase call represents the probability that one or more vehicles will place a call for service on the associated timer. The probability of max-out represents the probability that the phase will extend to the force-off setting and terminate, perhaps leaving some unserved vehicles on the intersection approach.

Exhibit 15-26 summarizes the output for intersection 1, as it relates to the movements assigned to each timer. Separate sections of output are shown in the exhibit for the left-turn, through, and right-turn movements. The assigned movement row identifies the movement (previously identified in Exhibit 15-24) assigned to each timer.

EXHIBIT 15-26. Intersection 1 Timer-Based Movement Output.

<table>
<thead>
<tr>
<th>Timer Data</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left-Turn Movement Data</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Assigned Movement</td>
<td>1</td>
<td>0</td>
<td>3</td>
<td>0</td>
<td>5</td>
<td>0</td>
<td>7</td>
<td>0</td>
</tr>
<tr>
<td>Mvmt. Sat Flow, veh/h</td>
<td>1710.0</td>
<td>0.00</td>
<td>1710.0</td>
<td>0.00</td>
<td>1710.0</td>
<td>0.00</td>
<td>1710.0</td>
<td>0.00</td>
</tr>
<tr>
<td>Queue Serve Time g_s, s</td>
<td>10.802</td>
<td>0.00</td>
<td>4.613</td>
<td>0.00</td>
<td>11.435</td>
<td>0.00</td>
<td>4.613</td>
<td>0.00</td>
</tr>
<tr>
<td>Through Movement Data</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Assigned Movement</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>4</td>
<td>0</td>
<td>6</td>
<td>0</td>
<td>8</td>
</tr>
<tr>
<td>Mvmt. Sat Flow, veh/h</td>
<td>0.00</td>
<td>3600.0</td>
<td>0.00</td>
<td>3221.05</td>
<td>0.00</td>
<td>3600.0</td>
<td>0.00</td>
<td>3221.05</td>
</tr>
<tr>
<td>Queue Serve Time g_s, s</td>
<td>0.000</td>
<td>16.312</td>
<td>0.000</td>
<td>14.803</td>
<td>0.000</td>
<td>21.876</td>
<td>0.000</td>
<td>14.803</td>
</tr>
<tr>
<td>Right-Turn Movement Data</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Assigned Movement</td>
<td>0</td>
<td>12</td>
<td>0</td>
<td>14</td>
<td>0</td>
<td>16</td>
<td>0</td>
<td>18</td>
</tr>
<tr>
<td>Mvmt. Sat Flow, veh/h</td>
<td>0.00</td>
<td>1530.0</td>
<td>0.00</td>
<td>322.11</td>
<td>0.00</td>
<td>1530.0</td>
<td>0.00</td>
<td>322.11</td>
</tr>
<tr>
<td>Queue Serve Time g_s, s</td>
<td>0.000</td>
<td>0.347</td>
<td>0.000</td>
<td>14.803</td>
<td>0.000</td>
<td>0.522</td>
<td>0.000</td>
<td>14.803</td>
</tr>
</tbody>
</table>
The movement saturation flow rate shown in Exhibit 15-26 represents the saturation flow rate computed for the movement. For through movements in exclusive lanes, the movement saturation flow rate is equal to the number of through lanes times the input saturation flow rate. For turn movements in exclusive lanes, the saturation flow rate is equal to the input number of turn lanes times the input saturation flow rate times the reciprocal of the default equivalency factor (i.e., $E_L = 1.05; E_R = 1.18$). These defaults can be input to the methodology.

For left-turn movements that share a lane with a through movement, the saturation flow rate is computed in the Queue Accumulation Polygon routine. Also, when a turn movement shares a lane with a through movement, the movement saturation flow rate represents a proportional allocation of the lane saturation flow rate to each movement based on the distribution of each movement type in the lane. Thus, if a shared lane has 18.5 percent right-turning vehicles, 81.5 percent through vehicles, and a saturation flow rate of 1743 veh/h, then the turn movement saturation flow rate is 322 veh/h ($= 0.185 \times 1743$) and the through movement saturation flow rate is 1421 veh/h ($= 0.815 \times 1743$). If the through movement is served by one exclusive through lane plus the aforementioned shared lane, then the through movement saturation flow rate is 3221 veh/h ($= 1421 + 1800$).

The queue service time shown in Exhibit 15-26 is an output of the Queue Accumulation Polygon routine. It represents the time required for the queue formed by the red indication to clear the approach.

Exhibit 15-27 summarizes the output for intersection 1, as it relates to the left-turn lane group served by the various timers. The lane assignment row shows the letter “L” as a reminder that the timer is serving a left-turn lane group. Other letter combinations are possible. For example, “L+T” would indicate the timer is serving a lane group consisting of a shared lane serving left-turn and through movements. A “L+T+R” sequence indicates a single lane approach serving all movements. For left-turn movements, the operational mode is also indicated. “Prot” indicates a protected left-turn mode. “Pr/Pm” indicates a protected-permissive left-turn mode. Any other designation with the letter “L” indicates either a permissive mode or split phasing.

**EXHIBIT 15-27. Intersection 1 Timer-Based Left Lane Group Output.**

<table>
<thead>
<tr>
<th>Timer Data</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Left Lane Group Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Assigned Movement</td>
<td>1</td>
<td>0</td>
<td>3</td>
<td>0</td>
<td>5</td>
<td>0</td>
<td>7</td>
<td>0</td>
</tr>
<tr>
<td>Lane Assignment</td>
<td>L (Prot)</td>
<td>L (Prot)</td>
<td>L (Prot)</td>
<td>L (Pr/Pm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lanes in Group</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Group Volume, veh/h</td>
<td>189.86</td>
<td>0.00</td>
<td>100.00</td>
<td>0.00</td>
<td>200.00</td>
<td>0.00</td>
<td>100.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Group Sat Flow, veh/h/ln</td>
<td>1710.00</td>
<td>0.00</td>
<td>1710.00</td>
<td>0.00</td>
<td>1710.00</td>
<td>0.00</td>
<td>1710.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Perm LT Sat Flow Rate s_p, veh/h/ln</td>
<td>0.00</td>
<td>0.00</td>
<td>871.14</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>871.14</td>
<td>0.00</td>
</tr>
<tr>
<td>Shared LT Sat Flow s_sh, veh/h/ln</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Perm LT Serve Time g_u, s</td>
<td>0.00</td>
<td>0.00</td>
<td>4.64</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>4.64</td>
<td>0.00</td>
</tr>
<tr>
<td>Perm LT Que Serve Time g_ps, s</td>
<td>0.00</td>
<td>0.00</td>
<td>1.92</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1.92</td>
<td>0.00</td>
</tr>
<tr>
<td>Time to first Blk g_f, s</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Serve Time pre Blk g_fs, s</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Portion LT inside lane, P_L</td>
<td>1.000</td>
<td>0.000</td>
<td>1.000</td>
<td>0.000</td>
<td>1.000</td>
<td>0.000</td>
<td>1.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Deterministic Delay, s/veh</td>
<td>41.086</td>
<td>0.000</td>
<td>30.534</td>
<td>0.000</td>
<td>42.205</td>
<td>0.000</td>
<td>30.534</td>
<td>0.000</td>
</tr>
<tr>
<td>Available Capacity, veh/h</td>
<td>500.59</td>
<td>0.000</td>
<td>403.10</td>
<td>0.000</td>
<td>500.59</td>
<td>0.000</td>
<td>403.10</td>
<td>0.000</td>
</tr>
<tr>
<td>Incremental Delay, s/veh</td>
<td>0.140</td>
<td>0.000</td>
<td>0.118</td>
<td>0.000</td>
<td>0.192</td>
<td>0.000</td>
<td>0.118</td>
<td>0.000</td>
</tr>
<tr>
<td>Incremental Stop Rate, stops/veh</td>
<td>0.027</td>
<td>0.000</td>
<td>0.023</td>
<td>0.000</td>
<td>0.037</td>
<td>0.000</td>
<td>0.023</td>
<td>0.000</td>
</tr>
</tbody>
</table>

The rows listed in Exhibit 15-27 that start with the permissive left-turn saturation flow rate and end with the deterministic delay are computed by the Queue Accumulation Polygon routine. The deterministic stop rate is also computed by this routine. The permissive left-turn saturation flow rate represents the filter saturation flow rate of a permissive left-turn movement. It is based on the opposing flow rate and is computed using the equation provided in the notes to Exhibit C16-3 in Chapter 16.

The shared left-turn saturation flow rate is the saturation flow rate of a shared left-turn and through lane during the period after the first blocking left-turn vehicle arrives but before the queue service ends. This flow rate is applicable when the opposing approach has one traffic lane. It reflects the opportunities to serve the subject approach that are created by left-turn vehicles in the opposing lane.

The permissive left-turn service time represents the time available for permissive left-turns. In general, it is the time in the opposing through movement phase that begins with the start of its green extension time and ends with the end of the green interval. However, its duration can vary depending on phase sequence and timing.
The permissive left-turn queue service time represents the time required to serve the left-turn queue. This time occurs during the phase in which the left-turn movement is served permissively. It exists for phases that operate in the permissive or the protected-permissive mode.

The time to first block is applicable to a lane group with a shared lane and a left-turn movement that operates in the permissive mode. It represents the time from the start of the through phase until the first left-turn vehicle arrives to the stop line and stops to wait for an acceptable gap in oncoming traffic. This variable is computed using Equation C16-10 or C16-11 from Chapter 16.

The queue service time prior to the first block represents the queue service time for a stream of through movements in a shared left-turn and through lane. If the left-turn volume is low, the time to first block may occur well into the phase. In this case, it is possible that the queue of through vehicles in the shared lane will be served before the first left-turn vehicle arrives. This variable applies only to lane groups with a shared left-turn lane.

The proportion of left-turn vehicles in the inside lane represents the distribution of vehicles in the inside lane. If a left-turn bay exists, then it is the “inside” lane and the proportion equals 1.0.

The deterministic delay represents the area under the queue accumulation polygon. This polygon is based on an average arrival rate during the green indication and an average arrival rate during the red indication. As such, it reflects the effect of progression on the delay estimate (i.e., the product of uniform delay and the progression adjustment factor).

The available capacity is computed during the Incremental Delay routine. For non-coordinated phases (i.e., all phases except phases 2 and 6), it is based on the equivalent maximum green. For coordinated phases, the available capacity is computed using the average effective green time in a manner consistent with the estimation of capacity for a pretimed phase.

The incremental delay is computed using the Incremental Delay routine. For non-coordinated phases, it uses the available capacity estimate. For coordinated phases, it uses the phase capacity calculated using traditional methods.

The deterministic stop rate is computed during the Queue Accumulation Polygon routine. This polygon is based on the specification of arrival rates during the red and green intervals. As such, it reflects the effect of progression on stop rate.

The incremental stop rate is computed during the Overflow Stop Rate routine. It uses available capacity to compute the overflow stops for the non-coordinated phases. It uses the traditional calculation of capacity to estimate the incremental stop rate for the coordinated phases.

Exhibit 15-28 summarizes the output for intersection 1, as it relates to the through and the right-turn lane groups.

<table>
<thead>
<tr>
<th>Timer Data</th>
<th>Timer:</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Middle Lane Group Data</strong></td>
<td>Assigned Movement</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>4</td>
<td>0</td>
<td>6</td>
<td>0</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Lane Assignment</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
</tr>
<tr>
<td></td>
<td>Lanes in Group</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Group Volume, veh/h</td>
<td>0.00</td>
<td>1000.00</td>
<td>0.00</td>
<td>279.41</td>
<td>0.00</td>
<td>949.29</td>
<td>0.00</td>
<td>279.41</td>
</tr>
<tr>
<td></td>
<td>Group Sat Flow, veh/h/ln</td>
<td>0.00</td>
<td>1800.00</td>
<td>0.00</td>
<td>1800.00</td>
<td>0.00</td>
<td>1800.00</td>
<td>0.00</td>
<td>1800.00</td>
</tr>
<tr>
<td></td>
<td>Deterministic Delay, s/veh</td>
<td>0.000</td>
<td>12.764</td>
<td>0.000</td>
<td>38.411</td>
<td>0.000</td>
<td>25.549</td>
<td>0.000</td>
<td>38.411</td>
</tr>
<tr>
<td></td>
<td>Available Capacity, veh/h</td>
<td>0.000</td>
<td>1701.70</td>
<td>0.000</td>
<td>570.87</td>
<td>0.000</td>
<td>1677.91</td>
<td>0.000</td>
<td>570.87</td>
</tr>
<tr>
<td></td>
<td>Incremental Delay, s/veh</td>
<td>0.000</td>
<td>1.495</td>
<td>0.000</td>
<td>0.242</td>
<td>0.000</td>
<td>1.104</td>
<td>0.000</td>
<td>0.242</td>
</tr>
<tr>
<td></td>
<td>Deterministic Stop Rate, stops/veh</td>
<td>0.000</td>
<td>0.355</td>
<td>0.000</td>
<td>0.811</td>
<td>0.000</td>
<td>0.728</td>
<td>0.000</td>
<td>0.811</td>
</tr>
<tr>
<td></td>
<td>Incremental Stop Rate, stops/veh</td>
<td>0.000</td>
<td>0.055</td>
<td>0.000</td>
<td>0.048</td>
<td>0.000</td>
<td>0.042</td>
<td>0.000</td>
<td>0.048</td>
</tr>
<tr>
<td><strong>Right Lane Group Data</strong></td>
<td>Assigned Movement</td>
<td>0</td>
<td>12</td>
<td>0</td>
<td>14</td>
<td>0</td>
<td>16</td>
<td>0</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>Lane Assignment</td>
<td>R</td>
<td>R</td>
<td>T+R</td>
<td>R</td>
<td>T+R</td>
<td>T+R</td>
<td>T+R</td>
<td>T+R</td>
</tr>
<tr>
<td></td>
<td>Lanes in Group</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Group Volume, veh/h</td>
<td>0.00</td>
<td>10.00</td>
<td>0.00</td>
<td>270.59</td>
<td>0.00</td>
<td>9.49</td>
<td>0.00</td>
<td>270.59</td>
</tr>
<tr>
<td></td>
<td>Group Sat Flow, veh/h/ln</td>
<td>0.00</td>
<td>1530.00</td>
<td>0.00</td>
<td>1743.16</td>
<td>0.00</td>
<td>1530.00</td>
<td>0.00</td>
<td>1743.16</td>
</tr>
<tr>
<td></td>
<td>Portion RT outside lane, P_R</td>
<td>0.000</td>
<td>1.000</td>
<td>0.000</td>
<td>0.185</td>
<td>0.000</td>
<td>1.000</td>
<td>0.000</td>
<td>0.185</td>
</tr>
<tr>
<td></td>
<td>Deterministic Delay, s/veh</td>
<td>0.000</td>
<td>13.994</td>
<td>0.000</td>
<td>38.411</td>
<td>0.000</td>
<td>22.644</td>
<td>0.000</td>
<td>38.411</td>
</tr>
<tr>
<td></td>
<td>Available Capacity, veh/h</td>
<td>0.000</td>
<td>723.22</td>
<td>0.000</td>
<td>552.84</td>
<td>0.000</td>
<td>713.11</td>
<td>0.000</td>
<td>552.84</td>
</tr>
<tr>
<td></td>
<td>Incremental Delay, s/veh</td>
<td>0.000</td>
<td>0.035</td>
<td>0.000</td>
<td>0.025</td>
<td>0.000</td>
<td>0.027</td>
<td>0.000</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td>Deterministic Stop Rate, stops/veh</td>
<td>0.000</td>
<td>0.041</td>
<td>0.000</td>
<td>0.011</td>
<td>0.000</td>
<td>0.065</td>
<td>0.000</td>
<td>0.011</td>
</tr>
<tr>
<td></td>
<td>Incremental Stop Rate, stops/veh</td>
<td>0.000</td>
<td>0.049</td>
<td>0.000</td>
<td>0.048</td>
<td>0.000</td>
<td>0.039</td>
<td>0.000</td>
<td>0.048</td>
</tr>
</tbody>
</table>
Exhibit 15-29 illustrates the output statistics for the two active driveways located on Segment 1. The first six rows listed in the exhibit correspond to driveway 1 (D1) and the second six rows correspond to driveway 2 (D2) (as shown previously in the urban street schematic). Additional sets of six rows would be provided in this table if additional driveways were evaluated. The eastbound and westbound volumes listed in the exhibit are not equal to the input volumes. The change is due to the Volume Balance routine. These volumes were adjusted such that they equal the volume discharging from the upstream junction (the boundary signalized intersection). This routine achieves balance between all junction pairs (e.g., between driveway 1 and driveway 2, between driveway 2 and signal 2, etc.) on all segments.

**EXHIBIT 15-29. Segment 1 Movement-Based Access Point Output.**

<table>
<thead>
<tr>
<th>SEGMENT 1</th>
<th>EB</th>
<th>EB</th>
<th>EB</th>
<th>WB</th>
<th>WB</th>
<th>NB</th>
<th>NB</th>
<th>NB</th>
<th>SB</th>
<th>SB</th>
<th>SB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Movement:</td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
<td>LT</td>
<td>TH</td>
</tr>
<tr>
<td>1: Volume, veh/h</td>
<td>74.80</td>
<td>981.71</td>
<td>90.50</td>
<td>73.80</td>
<td>968.64</td>
<td>92.25</td>
<td>80.00</td>
<td>0.00</td>
<td>100.00</td>
<td>80.00</td>
<td>0.00</td>
</tr>
<tr>
<td>1: Lanes</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>1: Portion time blocked</td>
<td>0.210</td>
<td>0.000</td>
<td>0.000</td>
<td>0.170</td>
<td>0.000</td>
<td>0.000</td>
<td>0.380</td>
<td>0.380</td>
<td>0.380</td>
<td>0.380</td>
<td>0.380</td>
</tr>
<tr>
<td>1: Delay to through vehicles, s/veh</td>
<td>0.000</td>
<td>0.150</td>
<td>0.000</td>
<td>0.000</td>
<td>0.157</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>1: Prob. inside lane blocked by left</td>
<td>0.000</td>
<td>0.094</td>
<td>0.000</td>
<td>0.000</td>
<td>0.098</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>1: Dist. from West/South signal, ft</td>
<td>600</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2: Volume, veh/h</td>
<td>75.56</td>
<td>991.70</td>
<td>94.45</td>
<td>72.74</td>
<td>954.69</td>
<td>90.92</td>
<td>80.00</td>
<td>0.00</td>
<td>100.00</td>
<td>80.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2: Lanes</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>2: Portion time blocked</td>
<td>0.220</td>
<td>0.000</td>
<td>0.000</td>
<td>0.170</td>
<td>0.000</td>
<td>0.000</td>
<td>0.390</td>
<td>0.390</td>
<td>0.390</td>
<td>0.390</td>
<td>0.390</td>
</tr>
<tr>
<td>2: Delay to through vehicles, s/veh</td>
<td>0.000</td>
<td>0.147</td>
<td>0.000</td>
<td>0.000</td>
<td>0.155</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2: Prob. inside lane blocked by left</td>
<td>0.000</td>
<td>0.092</td>
<td>0.000</td>
<td>0.000</td>
<td>0.098</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2: Dist. from West/South signal, ft</td>
<td>1200</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The proportion of time blocked is computed during the routine of the same name. It represents the proportion of time during the cycle that the associated access point movement is blocked by the presence of a platoon passing through the intersection. For major-street left-turns, the platoon of concern approaches from the opposing direction. For the minor-street left-turn, platoons can approach from either direction and can combine to block this left-turn for extended time periods. This trend can be seen by comparing the proportion of time blocked for the eastbound (major-street) left-turn (i.e., 0.21) with that for the northbound (minor street) left-turn (i.e., 0.38) at driveway 1.

The delay to through vehicles is computed by the Delay Due to Turns module. It represents the sum of the delays due to vehicles turning left from the major street and the delay due to vehicles turning right from the major street. Compared to signalized intersection delays, the delay due to turns tends to be small. But, it can be significant enough to influence travel speed if there are several high volume access points on a street and only one or two through lanes in each direction of travel.

The probability of the inside lane being blocked is computed by the Delay Due to Left Turns routine. This probability indicates the probability that the left-turn bay at an access point will overflow into the inside through lane on the street segment. Hence, it indicates the potential for a through vehicle to be delayed by a left-turn maneuver. The street being evaluated has an undivided cross section and no left-turn bays are provided at the driveways. In this situation, the probability of overflow is relatively large.

Exhibit 15-30 summarizes the performance measures for each segment and the two-segment system. Also shown are the results from the Spillback Check routine. The first group of nine rows corresponds to segment 1. The second group corresponds to segment 2. The movements indicated in the column heading represent the movements exiting the corresponding segment. Thus, the westbound movements on segment 1 are those that occur at signalized intersection 1. Similarly, the eastbound movements on segment 1 are those that occur at signalized intersection 2.

The spillback check routine computes the time of spillback for each of the internal segment movements. The value “999.00” is recorded for a movement if this time exceeds 1000 h. It typically is an indication that the movement volume is less than its phase capacity. If several movements experience spillback (i.e., have a value less than 1000 h), then the time of first spillback is reported at the bottom of Exhibit 15-30. For the example street, none of the segments experience spillback.

The free-flow speed and running time statistics are computed by the Running Time routine. These calculations were previously illustrated in Example Problem 1. The values are very similar between the two examples because they are based on the same pair of street segments.

The through delay listed is computed as a weighted average delay for the lane groups serving through movements at the downstream boundary intersection. The weight used in this average is the volume of through vehicles served by the lane group.

<table>
<thead>
<tr>
<th>SEGMENT DATA</th>
<th>EB</th>
<th>EB</th>
<th>EB</th>
<th>WB</th>
<th>WB</th>
<th>WB</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
<td>LT</td>
<td>TH</td>
<td>RT</td>
</tr>
<tr>
<td>Seg.No.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bay/Lane</td>
<td>999.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ShrdLane</td>
<td>999.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base Free-Flow Speed, mph</td>
<td>40.78</td>
<td>40.78</td>
<td>36.69</td>
<td>36.71</td>
<td>12.194</td>
<td>20.43</td>
</tr>
<tr>
<td>Running Time, s</td>
<td>33.45</td>
<td>33.43</td>
<td>33.45</td>
<td>33.43</td>
<td>33.45</td>
<td>33.45</td>
</tr>
<tr>
<td>Running Speed, mph</td>
<td>36.69</td>
<td>36.71</td>
<td>36.69</td>
<td>36.71</td>
<td>36.69</td>
<td>36.71</td>
</tr>
<tr>
<td>Travel Speed, mph</td>
<td>26.89</td>
<td>26.89</td>
<td>26.89</td>
<td>26.89</td>
<td>26.89</td>
<td>26.89</td>
</tr>
<tr>
<td>Stop Rate, stops/veh</td>
<td>0.348</td>
<td>0.348</td>
<td>0.348</td>
<td>0.348</td>
<td>0.348</td>
<td>0.348</td>
</tr>
<tr>
<td>Spatial Stop Rate, stops/mi</td>
<td>1.02</td>
<td>2.26</td>
<td>1.02</td>
<td>2.26</td>
<td>1.02</td>
<td>2.26</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bay/Lane</td>
<td>999.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ShrdLane</td>
<td>999.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base Free-Flow Speed, mph</td>
<td>40.78</td>
<td>40.78</td>
<td>36.69</td>
<td>36.71</td>
<td>12.478</td>
<td>20.83</td>
</tr>
<tr>
<td>Running Time, s</td>
<td>33.43</td>
<td>33.48</td>
<td>33.43</td>
<td>33.48</td>
<td>33.48</td>
<td>33.48</td>
</tr>
<tr>
<td>Running Speed, mph</td>
<td>36.71</td>
<td>36.71</td>
<td>36.71</td>
<td>36.71</td>
<td>36.71</td>
<td>36.71</td>
</tr>
<tr>
<td>Travel Speed, mph</td>
<td>26.73</td>
<td>26.73</td>
<td>26.73</td>
<td>26.73</td>
<td>26.73</td>
<td>26.73</td>
</tr>
<tr>
<td>Stop Rate, stops/veh</td>
<td>0.355</td>
<td>0.355</td>
<td>0.355</td>
<td>0.355</td>
<td>0.355</td>
<td>0.355</td>
</tr>
<tr>
<td>Spatial Stop Rate, stops/mi</td>
<td>1.04</td>
<td>2.17</td>
<td>1.04</td>
<td>2.17</td>
<td>1.04</td>
<td>2.17</td>
</tr>
<tr>
<td>System Travel Time, s:</td>
<td>91.56</td>
<td>119.00</td>
<td>91.56</td>
<td>119.00</td>
<td>91.56</td>
<td>119.00</td>
</tr>
<tr>
<td>System Travel Speed, mph:</td>
<td>26.81</td>
<td>20.63</td>
<td>26.81</td>
<td>20.63</td>
<td>26.81</td>
<td>20.63</td>
</tr>
<tr>
<td>System Spatial Stop Rate, veh/mi:</td>
<td>1.03</td>
<td>2.22</td>
<td>1.03</td>
<td>2.22</td>
<td>1.03</td>
<td>2.22</td>
</tr>
</tbody>
</table>

The remaining statistics listed in Exhibit 15-30 are computed using the Performance Measures module. The calculations were previously illustrated in Example Problem 1. The system statistics represent an aggregation of the travel time and stop rate for both segments in both directions of travel.

Results. The calculations shown in Exhibit 15-30 indicate that the system travel speed for the eastbound direction is 26.8 mph and for the westbound direction is 20.6 mph. The eastbound and westbound system spatial stop rate is 1.03 and 2.22 stops/mi, respectively. The travel speed for the eastbound direction is 66 percent (=26.81/40.78 ×100) of the base free-flow speed and, according to Exhibit 15-14, corresponds to level of service “C.” The westbound level of service is similarly computed to be “C.”
V. REFERENCES


OVERVIEW

This appendix describes the methodology used for an operational analysis of a coordinated system. This methodology is computationally intense such that it cannot be easily described in equation form. Moreover, this intensity is likely to discourage manual applications of the methodology (i.e., with worksheets and a hand calculator). Therefore, it has been implemented as a computational engine using the macro support provided in a popular spreadsheet. It is envisioned that this spreadsheet (or third-party implementations of it) will be used to automate applications of the methodology.

The methodology is developed to evaluate coordinated-actuated urban street operation. Each street segment is bounded by a signalized intersection. Two-way stop-controlled intersections may exist along the length of the segment; with stop control used to regulate the cross-street or driveway movements.

The methodology is sufficiently general that it can also be used to evaluate coordinated-pretimed operation and non-coordinated operation where the boundary intersections have stop or yield control. Coordinated-pretimed operation is achieved by adjusting selected controller inputs such that the signal operation is effectively pretimed. Non-coordinated operation is evaluated using the worksheets provided in Part III - Applications.

The methodology is designed to integrate the analysis of traffic flow along the segment with the analysis of traffic flow at the intersections bounding the segment. The vehicle flow patterns exiting one segment are used to define the entry flow pattern to the next segment. In this manner, the methodology is able to accurately account for the effect of traffic progression along an urban street facility.

MODULE DESCRIPTION

This part of the appendix describes the individual modules that comprise the urban streets methodology. The description is provided at a level of detail sufficient to understand the computational objectives of each module and the flow of information during the evaluation of a segment. The software implementation of this methodology should be consulted for details about the individual calculations and equations.

The computational steps involved in an operational analysis of a coordinated signal system are outlined in Exhibit 15-23. The steps are embodied in five modules, with each module focused on a key component of the analysis. The five modules are:

- Demand Adjustment Module
- Segment Analysis Module
- Signalized Intersection Module
- Delay Due to Turns Module
- Performance Measures Module

Each of these modules is described in more detail in the following sections. A more complete description of these modules is provided in Reference 1.

Demand Adjustment Module

The Demand Adjustment module adjusts the input volumes such that they reasonably reflect actual operating conditions. These adjustments have no effect if existing traffic movement volumes are accurately quantified for the subject segment and all movements operate below their capacity. However, if the demand volume for one or more movements exceeds its capacity or if there is disagreement between the volume that enters and exits a segment, then some movement volumes will need to be adjusted to accurately evaluate the segment operation.
This section describes three routines that check the input volumes and make adjustments if necessary. These routines are:

- Capacity Constraint and Volume Balance Routine
- Origin-Destination Distribution Routine
- Spillback Check Routine

A brief overview of each routine is described in the following subsections.

**Capacity Constraint and Volume Balance Routine**

Initially, this routine compares the volumes entering the segment at the upstream boundary intersection with the associated phase capacity. If the volume exceeds the capacity, then the discharge volume is reduced such that it equals the phase capacity.

Phase capacity is dependent on phase duration, which is computed in a subsequent module. This relationship introduces a circularity in the computations that requires each module to be re-computed, with the current calculations based on information computed during the previous iteration. Convergence to the steady-state solution typically occurs within 15 iterations.

As a second step, this routine is to compare the total volume discharging the upstream intersection with the total volume exiting at each downstream junction (i.e., intersection or access point). If there is disagreement in the two volumes, the ratio of the two volumes is computed. This ratio is then used to adjust the exit (i.e., turn movement) volumes at the downstream location such that balance is achieved. The downstream volumes are adjusted (as opposed to the discharge volumes) to properly account for any capacity constraint imposed in the first step of this routine.

This routine does not check for capacity constraint due to “demand starvation.” Starvation occurs when a downstream phase is green but the upstream signal timing is such that no vehicles have been discharged in time to take advantage of the green indication. This type of capacity constraint occurs on short street segments with flows entering the segment from only one or two movements at the upstream intersection, such as at a signalized diamond interchange.

**Origin-Destination Distribution Routine**

This routine computes the origin-destination matrix that quantifies the distribution of each upstream discharge volume to the exit movements at each downstream junction. The algorithm used to compute this distribution is described in the section titled, Intersection Turning Movements in Chapter 10. The origin-destination matrix concept is described in the text associated with Exhibits 15-5 and 15-6.

**Spillback Check Routine**

This routine examines each of the traffic movements that exit the segment at the downstream boundary intersection. Spillback may occur if: (1) the movement volume exceeds its phase capacity and (2) the condition is sustained for a length of time sufficient for the queue to extend back to the upstream boundary intersection. This type of spillback is referred to as “sustained spillback.” This routine computes the time that spillback will occur for each exit movement. The movement that causes spillback to occur in the shortest time (if any) represents the controlling spillback event. Its time is reported by the computational engine for analyst consideration. The sequence of calculations for this routine is described in Reference 1.

This routine does not address “cyclic spillback.” This type of spillback is a result of queue growth during the red indication and can occur as often as every signal cycle. It is most likely to occur on short street segments with relatively long signal cycle lengths.

The capacity of one or more access point traffic movements can also be impeded by the presence of a queue from a downstream signalized intersection. However, this routine does not report the occurrence of this type of spillback. Most notably, the
duration of this impedance is not explicitly considered in the Delay Due to Turns module or the Proportion of Time Blocked routine.

**Segment Analysis Module**

The Segment Analysis module computes the flow profile of discharging vehicles at the upstream intersection, as is influenced by the signal timing and phase sequence. It uses this profile to compute the arrival flow profile at a downstream junction. This arrival flow profile is then compared with the downstream signal timing and phase sequence to compute the proportion of vehicles arriving during green. The arrival flow profile is also used to compute the proportion of time that a platoon blocks one or more traffic movements at a downstream access point. These two platoon descriptors are used in subsequent modules to compute delay.

This section describes five routines that are used to define the arrival flow profile and compute the related platoon descriptors. These routines are:
- Discharge Flow Profile Routine
- Running Time Routine
- Projected Arrival Flow Profile Routine
- Proportion Arriving During Green Routine
- Proportion of Time Blocked Routine

A brief overview of each routine is described in the following subsections.

**Discharge Flow Profile Routine**

A flow profile is a macroscopic representation of steady-state traffic flow conditions for the average signal cycle during the analysis period. The cycle is represented as a series of 1 s time intervals (hereafter referred to as “time steps”). The start time of the cycle is 0.0 s, relative to the system reference time. The time steps are numbered from 1 to $C^*$, where $C^*$ is the cycle length in units of time steps. The flow rate for step $i$ represents an average of the flows that occur during the time period corresponding to step $i$ for all cycles in the analysis period. This approach is conceptually the same as that used in the TRANSYT-7F model (2).

A discharge flow profile is computed for each of the upstream left-turn, through, and right-turn movements. Each profile is defined by the time that the signal is effectively green and by the time that the queue service time ends. During the queue service time, the discharge flow rate is equal to the saturation flow rate. After the queue service time is reached, the discharge rate is set equal to the “adjusted discharge volume.” The adjusted discharge volume is equal to the discharge volume computed during the previous module but adjusted to reflect the “proportion of arrivals during green.” This latter adjustment adapts the discharge flow pattern to reflect platoon arrivals on the upstream segment.

The saturation flow rate for a through movement served in an exclusive lane is an input to the methodology. In contrast, the saturation flow rate for a shared lane is computed in the Signalized Intersection Module.

The discharge flow profile is dependent on saturation flow rate, queue service time, phase duration, and proportion of arrivals during green for the discharging movements. These variables are computed in subsequent routines. This relationship introduces a circularity in the computations that requires an iterative sequence of calculations to converge on the steady-state solution.

**Running Time Routine**

The running time routine computes the running time between the upstream intersection and each downstream junction. The procedure embodied by this routine was previously described in Part II - Methodology. The sequence of calculations is described in the text associated with Exhibit 15-16.
One component of running time is the delay due to various mid-segment sources. One notable source of delay is left or right turns from the segment into an access point. This delay is computed in the Delay Due to Turns module. Other sources of delay include on-street parking maneuvers and pedestrian crosswalks. Delay from these sources represents an input variable to the methodology.

Running time is dependent on delay due to turns, which is computed in a subsequent module. This relationship introduces a circularity in the computations that requires an iterative sequence of calculations to converge on the steady-state solution.

**Projected Arrival Flow Profile Routine**

This subsection describes the procedure for predicting the arrival flow profile at a downstream junction. This flow profile is based on the discharge flow profile and running time computed previously. The discharge flow profile is used with a platoon dispersion model to compute the arrival flow profile. The platoon dispersion model is summarized in the next subsection. Then, the procedure for using this model to estimate the arrival flow profile is described in the second subsection.

**Platoon Dispersion Model.** The platoon dispersion model was originally developed by Robertson (3) for use in the TRANSYT model. Input to the model is the discharge flow profile for a specified traffic movement. Output statistics from the model include: (1) the arrival time of the leading vehicles in the platoon to a specified downstream junction and (2) the flow rate during each subsequent time step.

In general, the arrival flow profile has a lower peak flow rate than the discharge flow profile owing to the dispersion of the platoon as it travels down the street. Also, for similar reasons, the arrival flow profile is spread out over a longer period of time than the discharge flow profile. The rate of dispersion increases with increasing segment running time, as may be caused by access point activity, on-street parking maneuvers, and other mid-segment delay sources.

The platoon dispersion model is described by the following equation:

\[
q'_{a,u,j} = F q'_{u,i} + (1 - F) q'_{a,u,j-1}
\]

with,

\[
j = i + t'
\]

where,

- \(q'_{a,u,j}\) = arrival flow in time step \(j\) at a downstream intersection from upstream source \(u\), veh/step;
- \(q'_{u,i}\) = departure flow in time step \(i\) at upstream source \(u\), veh/step;
- \(F\) = smoothing factor;
- \(j\) = time step associated with platoon arrival time; and
- \(t'\) = platoon arrival time, steps.

The upstream flow source \(u\) can be either the left-turn, through, or right-turn movement at the upstream boundary intersection. It can also be the collective set of left-turn or right-turn movements at access points between the upstream boundary intersection and the subject junction.

Exhibit A15-1 illustrates an arrival flow profile obtained from Equation A15-1. In this figure, the discharge flow profile is input to the model as variable \(q'_{u,i}\). The dashed rectangles that form the discharge flow profile indicate the flow rate during each of nine time steps \((i = 1, 2, 3, ... 9)\) that are each \(d_t\) seconds in duration. The vehicles that depart in the first time step \((i = 1)\) arrive at the downstream intersection after traveling an amount of time equal to \(t'\) steps. The arrival flow at any time step \(j (= i + t')\) is computed using Equation A15-1.
Research (1) indicates that the following relationship exists between the smoothing factor and running time:

\[
F = \frac{1}{1 + 0.138 T'_R + 0.315/d_t}
\]  

(A15-3)

where,

- \(T'_R\) = segment running time \((= T_R/d_t)\), steps;
- \(d_t\) = time step duration, s/step.

The time step duration used in the methodology is 1.0 s/step. Shorter values can be rationalized to provide a more accurate representation of the profile, but they also increase the time required for the computations. Experience indicates that 1.0 s/step provides a good balance between accuracy and computation time.

The following equation is used to compute platoon arrival time:

\[
t' = T'_R = \frac{1}{F} + 1.25
\]  

(A15-4)

**Arrival Flow Profile.** This section describes the procedure for computing the arrival flow profile. Typically, there are three upstream traffic movements that depart at different times during the signal cycle; they are: minor street right turn, major street through, and minor street left turn. Traffic may also enter the segment at various mid-block access points. Exhibit A15-2 illustrates how these movements join to form the arrival flow profile for the subject junction.

In application, the discharge flow profile for each of the departing movements is obtained from the Discharge Flow Profile routine. These profiles are shown in the first of the three x-y plots in Exhibit A15-2. Then, the platoon dispersion model is used to estimate the arrival flows for each movement at a downstream intersection. These arrival flow profiles are shown in the second x-y plot in the exhibit. Although not shown, arrivals from mid-segment access points are assumed to have a uniform arrival flow profile (i.e., a constant flow rate for all time steps).

Finally, the origin-destination matrix is used to distribute each arrival flow profile to each of the downstream exit movements. The four arrival flow profiles associated with the subject exit movement are added together to produce the combined arrival flow profile. This profile is shown in the third x-y plot. The upstream movement contributions to this profile are indicated by arrows.
Comparison of the profiles in the first and second x-y plots of Exhibit A15-2 illustrates the platoon dispersion process. In the first x-y plot, the major street through movement has formed a dense platoon as it departs the upstream intersection. However, when this platoon reaches the downstream intersection it has spread out over time and has a lower peak flow rate. In general, the amount of platoon dispersion increases with increasing segment length. For very long segments, the platoon structure degrades and arrivals become uniform throughout the cycle (i.e., random).

In addition to the effect of dispersion, platoon structure can also degrade as a result of significant access point activity along the segment. Streets with frequent active access points tend to have more vehicles leave the platoon (i.e., turn from the segment at an access point) and enter the segment after the platoon passes (i.e., turn in to the segment at an access point). Both activities result in platoon decay.

The effect of platoon decay is modeled using the origin-destination matrix, where the combined access point activity is represented as one volume assigned to mid-segment origins and destinations. When the access point volume is large, it corresponds to a smaller volume that enters at the upstream boundary intersection as a defined platoon. This results in a larger portion of the combined arrival flow profile defined by uniform (rather than platoon) arrivals. When a street has busy access points, platoon decay tends to be a more dominant cause of platoon degradation than platoon dispersion.

**Proportion Arriving During Green Routine**

The combined arrival flow profile is used to estimate the proportion of vehicles that arrive during the effective green period of a specified phase. Any phase that controls an
exit movement at the boundary intersection can be considered. The manner in which this proportion is estimated from the combined arrival flow profile is shown in Exhibit A15-3. The exhibit illustrates the combined arrival flow profile for the major-street through lane group; however, a similar profile can be constructed for turn-related lane groups if they exist.


The gray shaded area in Exhibit A15-3 represents the volume arriving during the effective green time of the phase serving the through lane group. This volume is computed by summing the vehicle count represented by each time step that occurs during the effective green period. The proportion of vehicles arriving during the green period for a specified lane group is computed using the following equation.

\[ P = \frac{n_g}{v_d C} \]  

(A15-5)

where,

- \( P \) = proportion of vehicles arriving during green;
- \( n_g \) = arrival count during effective green, veh;
- \( v_d \) = lane group volume, veh/s; and
- \( C \) = cycle length, s.

Proportion of Time Blocked Routine

The combined arrival flow profile can be used to estimate the time that a dense platoon passes through a downstream access point. During this time period, the platoon is sufficiently dense as to preclude a minor movement driver from finding an acceptable gap.

The use of the arrival flow profile to estimate the blocked period duration is shown in Exhibit A15-4. The profile shown represents the combined arrival flow profile for the through lane group at a downstream access point. The dashed line represents the critical platoon flow rate \( q_c \). Flow rates in excess of this threshold are rationalized to be associated with platoon headways that are too short to be entered (or crossed) by minor movements. A critical platoon flow rate of 1000 veh/h (0.28 veh/s) is recommended.
In the situation where a driver desires to complete a left turn from the major street across the traffic stream represented by Exhibit A15-4, the proportion of time blocked is computed using the following equation. For this maneuver, the blocked period duration is based on the flow profile of the opposing through lane group.

\[
p_b = \frac{t'_p d_t}{C}
\]

where,
- \(p_b\) = proportion of time blocked;
- \(t'_p\) = blocked period duration, steps;
- \(d_t\) = time step duration, s/step; and
- \(C\) = cycle length, s.

Equation 15-6 is also used for the minor-street right-turn movement. However, the blocked period duration is computed for the through lane group approaching from the left. For the minor street left-turn and through movements, the arrival flow profiles from both directions are evaluated. In this instance, the blocked period duration represents the time when a platoon from either direction is present in the intersection.

**Signalized Intersection Module**

The objectives of the Signalized Intersection module are: (1) to compute the average phase duration, (2) to compute deterministic delay, and (3) to compute deterministic stop rate. The calculation of average phase duration requires an iterative calculation process because of interdependencies among signal timing and queue service time. In contrast, the delay and stop rate calculations do not share these interdependencies and are computed only during the last iteration of the process.

The aforementioned interdependencies require the computation of average duration for all signal phases at an intersection, not just the phases that serve the segment through lane groups. Hence, the discussion in this section necessarily addresses all phases and associated lane groups. With one exception, lane groups are defined herein using the lane group rules described in Chapter 16. For the special case of shared lane operation on multilane approaches, separate lane groups are formed for each shared lane and for any remaining exclusive through lanes.
This section describes the five main routines that are used to determine the average phase duration, and the delay and stops for each lane group. These routines are:

- Volume Computations Routine
- Queue Accumulation Polygon Routine
- Maximum Allowable Headway Routine
- Equivalent Maximum Green Routine
- Average Phase Duration Routine

A brief overview of each routine is described in the following subsections.

**Volume Computations Routine**

The Volume Computation routine quantifies the time rate of calls submitted to the controller by the detectors. There are two call rates that are computed for each signal phase. The first rate represents the flow rate of calls for green extension that arrive during the green interval. The second call rate represents the flow rate of calls for phase activation that arrive during the red indication.

The call rate to extend the green indication for a given phase is based on the volume in the lane groups served by the phase and whether the phase ends at the barrier. If the subject phase ends at a barrier and simultaneous gap-out is enabled, then its call rate is based on the lane groups it serves plus those groups served by the phase in the other ring that ends at the same time. The extending call rate parameter is computed as:

$$\lambda^* = \sum_{i=1}^{m} \lambda_i$$  \hspace{1cm} (A15-7)

with,

$$\lambda_i = \frac{\phi_i q_i}{1 - \Delta_i q_i}$$  \hspace{1cm} (A15-8)

$$\phi_i = e^{-b_i \Delta_i q_i}$$  \hspace{1cm} (A15-9)

where,

- $\lambda^*$ = total call rate parameter, veh/s;
- $\lambda_i$ = call rate parameter for lane group $i$, veh/s;
- $\phi_i$ = proportion of free (unbunched) vehicles in lane group $i$;
- $q_i$ = average flow rate of traffic stream in lane group $i$ ($= v_i / 3600$), veh/s;
- $v_i$ = volume for lane group $i$, veh/h;
- $\Delta_i$ = headway of bunched vehicle stream ($= 1.5$ s for single-lane lane group, $0.5$ s otherwise), s;
- $m$ = number of lane groups served during the phase; and
- $b_i$ = bunching factor ($= 0.6, 0.5, 0.8$ for lane groups with $1, 2$, and $3$ or more lanes, respectively).

The call rate to activate a phase is used to determine the probability that the phase is activated in the forthcoming cycle sequence. This rate is based on the volume of the traffic movements served by the phase and whether the phase is associated with dual entry. In general, the activation call rate is equal to the total volume of the movements served. However, if the subject phase is set for dual entry, then its activation call rate is modified by adding to it the activation call rate of both concurrent phases (i.e., those in the other ring between the same barrier pair).

**Queue Accumulation Polygon Routine**

The Queue Accumulation Polygon routine analytically models the polygon that describes the queue length for a lane group at any instant in time during the cycle. This polygon is used to determine the queue service time for each intersection lane group.

A queue accumulation polygon for a through movement is shown in Exhibit 15-9. Other polygon shapes are possible, depending on whether the lane group includes a
shared lane and whether the lane group serves a left-turn movement from an exclusive lane. For this latter case, the polygon shape will be dictated by the left-turn mode of operation (i.e., permissive, protected, protected/permisive, split) and phase sequence (i.e., lead or lag). Illustrative polygons are shown Appendix B of Chapter 16.

The Queue Accumulation Polygon routine consists primarily of several routines that are collectively used to estimate key variables associated with the polygon for a given phase. These routines are needed to model the complex interactions associated with left-turn mode, phase sequence, and shared lane operation. These interactions influence lane group saturation flow rate and the duration of service provided.

**Permissive Service Time Routine.** This routine is used to evaluate a left-turn movement that is served using the permissive or protected-­permissive mode. The permissive green time for a left-turn movement represents the time during the opposing through phase that occurs after its queue service and before its yellow onset. This time is available to the left-turn movement for permissive operation. If protected-­permissive operation exists and Dallas Display is provided, then the permissive service time is equal to the unqueued effective green time for the opposing through movement. If Dallas Display is not used, then the permissive service time is dependent on phase sequence, the duration of the shorter through phase, and the queue service time of the opposing through movement.

**Time to First Block by Left-Turn Vehicle Routine.** This routine is applicable when left turns are served using the permissive mode on a shared lane approach. The time to first block represents the time that lapses from the start of the permissive green to the arrival of the first left-turn vehicle at the stop line. During this time, through vehicles in the inside shared lane depart at the saturation flow rate. Guidance for computing this variable is provided in Appendix C of Chapter 16.

**Lane Group Volume Distribution Routine.** This routine computes the proportion of turns in each shared lane. The computations vary depending on whether the shared lane includes a left-turn or right-turn movement. For multilane approaches, the routine considers jointly the effects of left-turn and right-turn vehicles when they are served in a shared lane. Equation C16-7 in Chapter 16 is used to compute the proportion of turns in the shared left-turn lane. A variation of this equation is used to compute the proportion of turns in the shared right-turn lane.

**Lane Group Saturation Flow Rate Routine.** The queue service time for a lane group is highly dependent on its saturation flow rate. The input saturation flow rate is used for through movements from exclusive lanes. The input saturation flow rate for a protected turn movement from exclusive lanes is adjusted by multiplying it by one of the following adjustment factors:

\[
f_{LT} = \frac{1}{E_L} \quad f_{RT} = \frac{1}{E_R}
\]

where,
- \(f_{LT}\) = left-turn adjustment factor;
- \(f_{RT}\) = right-turn adjustment factor;
- \(E_L\) = through-car equivalent for protected left turns (= 1.05); and
- \(E_R\) = through-car equivalent for protected right turns (= 1.18).

For lanes shared by left-turn and through movements, the procedure described in Appendix C of Chapter 16 is generally used. However, the overall average saturation flow rate adjustment factor \(f_m\) described in this procedure is not computed. Instead, the saturation flow rate of the individual flow periods during the cycle is computed and retained for polygon development. As noted previously, a lane shared by the left-turn and through movements is designated as a separate lane group and its saturation flow rate calculated separately from that of any adjacent lanes. The concepts in Appendix C are extended to this calculation. The calculation of saturation flow rate for a lane shared by the right-turn and through movements is based on similar concepts.
Queue Service Time and Deterministic Delay from QAP Routine. This routine uses the information computed by the previous routines to define the queue accumulation polygon (QAP). This information is combined with the computed arrival rates during the red and green intervals to compute the queue profile for the average cycle. The queue service time is a key output of this routine. It represents the time elapsed from the start of the effective green period to the point where the queue has been served. The area under the queue profile represents the total deterministic delay incurred by the average driver.

It is assumed in the development of the queue service model that the detection design is sufficient to prevent the occurrence of premature phase gap out. This assumption is valid if the detection length and passage time in combination provide a maximum allowable headway of 2.5 s or more.

Deterministic Stop Rate from ADP Routine. This routine uses the information computed by the previous routines to define the arrival-departure polygon (ADP). This polygon shows, at any instant in time, the location of the back, and front, of queue in terms of its distance from the stop line. In contrast, the queue accumulation polygon shows only the length of the queue at any instant in time. Both polygons use the same interval time and saturation flow rate information for their construction. The arrival-departure polygon is more conducive to the estimation of deterministic stop rate.

An example arrival-departure polygon for an exclusive movement was shown previously in Exhibit 15-13. Other polygon shapes are possible, depending on whether the lane group includes a shared lane and whether the lane group serves a left-turn movement from an exclusive lane. For this latter case, the polygon shape will be dictated by the left-turn mode of operation (i.e., permissive, protected, protected-permissive, split) and phase sequence (i.e., lead or lag).

Maximum Allowable Headway Routine

The Maximum Allowable Headway routine computes the equivalent maximum allowable headway (MAH) for the lane group, or groups, served by a phase. The MAH represents the maximum time headway between successive calls for service without terminating the phase by gap out. It is a useful characteristic for describing the detection design and timing associated with a phase. The MAH is dependent on the number of detectors serving the lane group, the length of these detectors, and the average vehicle speed in the lane group.

The MAH for stop line detection can be calculated using the following equation:

\[
MAH = PT + \frac{L_{da} + L_v}{1.47 \times S_a}
\]  

where,

- \(MAH\) = maximum allowable headway, s;
- \(PT\) = passage time setting (sometimes referred to as vehicle interval, extension interval, or unit extension setting), s;
- \(L_{da}\) = length of the stop line detection zone, ft;
- \(L_v\) = detected length of vehicle, ft; and
- \(S_a\) = average speed on the intersection approach, mph.

If a phase serves two or more lane groups, then an equivalent MAH is calculated to describe the combined effect of the detection on green extension. This calculation is represented as a weighted average of the MAHs of the various lane groups, where the call rate parameter is used as the weighting factor. To illustrate, the following equation is used to calculate the equivalent MAH for an phase that serves a through lane group and a (permissive) left-turn lane group.

\[
MAH^* = \frac{\lambda_{th} MAH_{th} + \lambda_{lt} (MAH_{lt} + 3600/(S_v - t_v))}{\lambda_{th} + \lambda_{lt}}
\]  

(A15-12)
where,

\[ \text{MAH}^{*} = \text{equivalent maximum allowable headway, s}; \]
\[ \text{MAH}_{\text{th}} = \text{maximum allowable headway for the through lane group, s}; \]
\[ \text{MAH}_{\text{lt}} = \text{maximum allowable headway for the left-turn lane group, s}; \]
\[ \lambda_{\text{th}} = \text{call rate parameter for the through lane group, veh/s}; \]
\[ \lambda_{\text{lt}} = \text{call rate parameter for the left-turn lane group, veh/s}; \]
\[ s_{\text{lt}} = \text{saturation flow of permissive left turns, veh/h/ln}; \]
\[ t_{f} = \text{follow-up headway, s}. \]

The saturation flow rate term in Equation A15-12 is used to increase the \( \text{MAH} \) by an amount equal to the service time of the permissive left-turn vehicle. This vehicle will stop on the detector while it waits for a gap in the opposing traffic stream. This behavior effectively extends the green interval for the phase by a corresponding amount.

The equivalent \( \text{MAH} \) is also calculated as a weighted average for phases that (1) end at a barrier and (2) are specified in the controller as having to gap out simultaneously (the default setting in most controllers) with a phase in the other ring. In this instance, the equivalent \( \text{MAH} \) is computed as a weighted average of all lane groups served by the terminating phases.

**Equivalent Maximum Green Routine**

In coordinated-actuated operation, the force-off points are used to constrain the duration of the actuated (non-coordinated) phases. The maximum green setting is also available to provide additional constraint; however, it is not commonly used. In fact, the default mode in most modern controllers is to inhibit the maximum green timer when the controller is used in a coordinated signal system.

The relationship between the force-off points, yield point, and phase splits is shown in Exhibit A15-5. The yield point is associated with the coordinated phases (i.e., phases 2 and 6). It coincides with the start of the change period (i.e., yellow change plus red clearance intervals). If a call for service by one of the non-coordinated phases arrives prior to the yield point, then the coordinated phases begin the termination process by presenting the yellow indication. If a call has not arrived by the yield point, the coordinated phase will remain in its green interval.

A permissive period typically follows the yield point. If a conflicting call arrives during the permissive period, then the phase termination process begins immediately and all phases associated with conflicting calls are served in sequence. Permissive periods are typically sufficiently long as to ensure that all calls for service are met during the signal cycle. The urban streets methodology does not explicitly model permissive periods. It is assumed that the permissive period begins at the yield point and is sufficiently long that all conflicting calls are served in sequence each cycle.

One force-off point is associated with each of the concurrent phase pairs 3 and 7, 4 and 8, and 1 and 5. If either phase of a pair is extended to its force-off point, the phase begins the termination process by presenting the yellow indication. Modern controllers compute the force-off points and yield point using the entered phase splits and change periods based on the relationships shown in Exhibit A15-5.

The concept of “equivalent” maximum green interval duration is useful for modeling the non-coordinated phase operation. The Equivalent Maximum Green routine computes an equivalent maximum green \( G_{\text{max}} \) for each non-coordinated phase. Its purpose is to replicate the effect of the force-off point on the phase’s green duration.

The procedure for estimating the equivalent maximum green varies by force mode. If the controller is set to operate with a floating force mode, each non-coordinated phase has its force-off point set at the split time after the phase first becomes active. The force-off point for a phase is established when the phase is first activated. Thus, the force-off point “floats,” or changes, each time the phase is activated. This operation allows unused split time to revert to the coordinated phase via an early return to green.
The equivalent maximum green for this mode is computed as being equal to the phase split less the change period. This relationship is shown in Exhibit A15-5 for phases 4 and 8.

**EXHIBIT A15-5. Force-Off Points, Yield Point, and Phase Splits.**

If the controller is set to operate with a fixed force mode, each non-coordinated phase has its force-off point set at a fixed time in the cycle, relative to time zero on the system master. The force-off points are established whenever a new timing plan is selected (e.g., by time of day) and remains “fixed” until a new plan is selected. The equivalent maximum green for a given phase equals the difference between its force-off point and the sum of the previous non-coordinated phases. This operation allows unused split time to revert to the next phase.

**Average Phase Duration Routine**

The duration of an actuated (non-coordinated) phase is comprised of four time periods. The first period represents the time lost while the queue reacts to the signal indication changing to green. The second interval represents the time required to clear the queue of vehicles. The third period represents the time the green is extended by arriving vehicles. It ends when there is a gap in traffic (i.e., gap out) or the green extends to the maximum limit (i.e., max out). The last period represents the change period. The duration of an actuated phase can be expressed by the following equation:

\[
D_p = l_1 + g_s + g_e + Y
\]  

(A15-13)

where,

- \(D_p\) = phase duration, s;
- \(l_1\) = start-up lost time, s;
- \(g_s\) = queue service time, s;
- \(g_e\) = green extension time, s; and
- \(Y\) = combined yellow change and red clearance interval duration, s.
The relationship between the variables in Equation A15-13 was shown previously in Exhibit 15-9 using a queue accumulation polygon.

The Average Phase Duration routine computes the average phase duration by first calculating the green extension time for each non-coordinated phase. It then uses the phase sequence and cycle length to compute the average duration of the coordinated phases. Queue service time for the non-coordinated phases was computed previously in the Queue Accumulation Polygon routine.

**Green Extension Time Routine.** The Green Extension Time routine combines the total call rate parameter, equivalent MAH, and equivalent maximum green from previous routines to estimate the phase extension time. This time is computed using the following equation:

\[
G_e = \frac{G^2 (1 - p^*)}{qp^* (1 - p)}
\]  

(A15-14)

with,

\[
p = 1 - \varphi^* e^{\Delta^* (MAH^* - \Delta^*)}
\]  

(A15-15)

\[
\varphi^* = e^{-\sum_{i=1}^{n} b_i \Delta_i}
\]  

(A15-16)

\[
\Delta^* = \frac{\sum_{i=1}^{n} \lambda_i \Delta_i}{\lambda^*}
\]  

(A15-17)

\[
n = qp^* \left[ G_{\text{max}} - (G^*_t + l_i) \right] \geq 0.0
\]  

(A15-18)

\[
q_p^* = \frac{1}{\Delta^* + 1/\lambda^*}
\]  

(A15-19)

where,

- \( p \) = probability of a call headway being less than the MAH;
- \( q_p^* \) = call flow rate of free (unbunched) traffic stream, veh/s;
- \( \varphi^* \) = combined proportion of free (unbunched) vehicles;
- \( \Delta^* \) = equivalent headway of bunched vehicle stream, s;
- \( G_{\text{max}} \) = equivalent maximum green for the subject phase, s; and
- \( n \) = number of calls necessary to extend the green to max out.

**Unbalanced Green Duration.** This routine computes the average green duration for each actuated phase.

\[
G_u = \text{larger of:} \left\{ \frac{l_i + G_s + G_e}{G_{\text{min}}} \left( 1 - e^{-\varphi^* C} \right) \right\} \leq G_{\text{max}}
\]  

(A15-20)

where,

- \( G_u \) = unbalanced green duration, s;
- \( q_p \) = phase activation call flow rate (= \( \Sigma q_i \)), veh/s; and
- \( G_{\text{min}} \) = minimum green setting for the subject phase, s.

The first term in Equation A15-20 ensures that every phase that is called has a green indication that equals or exceeds the minimum green setting. The second term in this equation represents the probability that a phase is called (i.e., not skipped). Thus, the green duration predicted by Equation A15-20 can have a value shorter than the minimum setting when there are frequent skipped phases.

**Phase Balance Routine.** This routine computes the average duration of the coordinated and non-coordinated phases. As a first step, the average green “demand” is computed for the non-coordinated phases. This demand represents the sum of the green
service time and green extension after conversion to an expected value based on the probability of the phase being called and with consideration of the minimum and maximum green values.

For the second step of this routine, the non-coordinated phases serving the minor street are adjusted such that phases in ring 1 equal the duration of phases in ring 2. This adjustment is shown in Exhibit A15-6. The time elapsed from the start of phase 2 to the end of phase 4 equals one signal cycle (i.e., 120 s). The duration of each phase is represented by its horizontal width, the value of which is indicated in the bottom portion of each phase block. For example, phase 4 has a duration of 42 s. The grey shaded portion of each phase block is an indication of the average green demand.

The following steps apply to achieve balance for the minor street. First, the duration of the actuated phase that occurs first in each pair is equated to its average green demand. Second, the total duration of each pair is computed and the longer pair dictates the duration of the concurrency group. Third, the second phase of the shorter pair is increased such that the total duration of phases 3 and 4 equals the total duration of phases 7 and 8. For example, the total duration of phases 3 and 4 is 60 s (= 18 + 42). This duration is longer than the 48 s needed to serve phases 7 and 8, so the duration of the second phase (i.e., phase 8) in this latter pair is increased to 36 s.

For the fourth step of this routine, the adjusted minor street phases are added with the major-street left-turn phases (if they exist) and subtracted from the cycle length to determine the time available to each coordinated phase. The rules for making this determination are based on the phase sequence for the major-street. For example, the major-street phases shown in Exhibit A15-6 indicate that a left-turn phase exists on each approach. In this situation, the phase duration for each left-turn phase is equal to its average green demand (from step 1). The duration of the coordinated phase is equal to the cycle length less the left-turn phase in the same ring and less the 60 s allocated to the minor street phases. The end result of this step is that the phases in each ring add to the cycle length.

**Delay Due to Turns Module**

The Delay Due to Turns module is used to compute the delay to through vehicles that follow vehicles turning from the major street into an unsignalized access point. This delay can be incurred at any access point along the street. For right-turn vehicles, the
delay results when the following vehicles’ speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane.

This section describes three routines that are used to quantify the delay due to turns at an access point. These routines are:

- Lane Volume Distribution Routine
- Delay Due to Left Turns Routine
- Delay Due to Right Turns Routine

A brief overview of each routine is described in the following subsections.

**Lane Volume Distribution Routine**

The Lane Volume Distribution routine is used initially in this module to determine the volume in each lane on the major-street approach to the access point. Through vehicles in the inside lane may incur delay if there is a left-turn maneuver. Through vehicles in the outside lane may incur delay if there is a right-turn maneuver. In anticipation of these potential delays, through drivers are predisposed to choose a lane that minimizes these delays.

The algorithm used in this routine was developed initially by Bonneson and McCoy (4). It is based on the assumption that drivers will minimize their travel time through the intersection or access point. This minimum is achieved when each approach lane has the same volume-to-saturation ratio. Mathematically, it results in the following equation for estimating the proportion of left-turn vehicles in the inside lane:

\[
P_L = \frac{\sum_{i=1}^{n} s_i (1 + P_L I_i)}{1 + P_L (E_L - 1) + P_L E_L I_i}
\]

(A15-21)

with,

\[
s_1 = \frac{s_L (1 + P_L I_L)}{1 + P_L (E_L - 1) + P_L E_L I_L}
\]

(A15-22)

\[
s_n = \frac{s_t}{1 + P_R (E_R - 1)}
\]

(A15-23)

\[
s_{s,...,n-1} = s_t
\]

(A15-24)

where,

- \( P_L \) = proportion of left-turn vehicles in the inside lane (i.e., lane 1);
- \( P_R \) = proportion of right-turn vehicles in the outside lane (i.e., lane \( n \));
- \( P_{LT} \) = proportion of left-turn vehicles in the approach flow;
- \( P_{RT} \) = proportion of right-turn vehicles in the approach flow;
- \( s_i \) = saturation flow rate in lane \( i \) (\( i = 1, 2, ..., n \)), veh/h/ln;
- \( s_t \) = saturation flow rate for a through traffic stream, veh/h/ln;
- \( E_L \) = through-car equivalent for a left-turn vehicle;
- \( E_R \) = through-car equivalent for a right-turn vehicle;
- \( I_i \) = indicator variable (= 1.0 when left-turning vehicles block the inside lane, 0.0 otherwise); and
- \( n \) = number of through lanes on the approach.

After \( P_L \) and \( P_R \) are computed from Equation A15-21, they are used in the following equations to estimate the volume in the inside and outside lanes:

\[
v_L = \frac{v_L}{P_L} \quad v_R = \frac{v_R}{P_R}
\]

(A15-25)
where,
\( v_1 \) = volume in the inside lane (i.e., lane 1), veh/h;
\( v_n \) = volume in the outside lane (i.e., lane \( n \)), veh/h;
\( v_L \) = left-turn volume on the approach, veh/h; and
\( v_R \) = right-turn volume on the approach, veh/h.

The algorithm was derived such that two traffic conditions could be represented. One condition is described as “blocked” and occurs when the inside through lane is occupied by one or more queued left-turn vehicles. When this condition exists, arriving through drivers often attempt to merge into an adjacent through lane. Many through drivers are able to complete this merge without stopping. On the other hand, those drivers that stop have to wait until the turn vehicle departs or until there is an opportunity to merge into the adjacent lane.

The second condition is described as “unblocked.” It exists whenever the inside through lane is not blocked by a left-turn queue. While this condition exists, the number of vehicles in each lane may not be exactly equal because some through drivers will choose to avoid the “threat” of delay due to a possible turning vehicle by choosing a lane where the presence of turning vehicles is negligible.

The algorithm was subsequently extended by Bonneson and Fitts (5). They noted that the original version of the model overestimated the through lane flow rate in the outside lane under very low and very high flow rate conditions. The extended version estimates the probability of a lane change, as influenced by the flow rate in the adjacent lane, and uses this information to adjust the estimated lane volumes.

**Delay Due to Left Turns Routine**

Through vehicles on the major-street approach to an unsignalized access point can incur delay when the left-turn queue exceeds the available storage and blocks the adjacent through lane (in this context, the undivided cross section is considered a major street approach having no left-turn storage). The through vehicles that follow are delayed when they stop behind the queue of turning vehicles. This delay ends when the left-turn vehicle departs or the through vehicle merges into the adjacent through lane. By merging into the adjacent lane, drivers reduce their delay relative to the delay they would have incurred had they waited for the left-turn queue to clear. This delay is computed as:

\[
d_{ap,1} = P_{ov} \left( \frac{1}{P_L} - 1 \right) \left( \frac{P_{LT}}{1 - P_{LT} - P_{RT}} \right) \tag{A15-26}
\]

where,
\( d_{ap,1} \) = through vehicle delay due to left turns, s/veh;
\( P_{ov} \) = probability of left-turn bay overflow (i.e., a queue in the inside through lane);
\( d_{i,1} \) = average delay to through vehicles in lane 1 (i.e., the inside lane), s/veh.

The average delay to through vehicles in lane 1 is determined to be the smaller of:
1. the delay incurred waiting for the left-turn vehicle to clear and
2. the delay incurred waiting for a gap in the adjacent lane in which to merge. These delays are computed using a time-dependent delay equation, similar to that used in Chapter 17. The probability of bay overflow is computed using the following equation:

\[
P_{ov} = \left( \frac{v_a P_{LT}}{c_i} \right)^{N_v} \tag{A15-27}
\]

where,
\( v_o \) = approach flow rate on the subject approach, veh/h;
\( c_i \) = left-turn capacity, veh/h;
\( N_v \) = number of left-turn vehicles that can store before spilling back into the inside through lane, veh.
For an undivided cross section, the number of left-turn vehicles that can store $N_l$ is equal to 0.0.

**Delay Due to Right Turns Routine**

A vehicle turning right from the major street into an access point often delays the through vehicles that follow it. Through vehicles are delayed because they have to reduce speed to avoid a collision with the vehicle ahead, the first of which has reduced speed to avoid a collision with the right-turning vehicle. This delay can be several seconds in duration for the first few through vehicles but will always decrease to negligible values for subsequent vehicles as the need to reduce speed diminishes. For purposes of running time calculation, this delay must be averaged over all through vehicles traveling in the subject direction. The resulting average delay is computed as:

$$d_{ap,r} = 0.67 d_{vr} \frac{P_{RT}}{1 - P_{LT} - P_{RT}}$$  \hspace{1cm} \text{(A15-28)}$$

with,

$$d_{vr} = \sum d_i \times P(h, \text{ short enough to be delayed}) \times P(i \text{ through veh.})$$  \hspace{1cm} \text{(A15-29)}$$

where,

$d_{ap,r}$ = through vehicle delay due to right turns, $s$/veh; and

$d_{vr}$ = through vehicle delay per right-turn maneuver, $s$/veh.

The through vehicle delay per right-turn maneuver is modeled using a time-space representation of traffic flow on a street (6). This type of representation is shown in Exhibit A15-7. The trajectories of one right-turn vehicle $R$ and three through vehicles $T$ are shown, where each vehicle has the same speed $u$. The trajectories of the through vehicles are sequentially evaluated to determine the delay $d$ each vehicle incurs as a consequence of the turn. The right-turn vehicle initiates the slowing process in the outside through lane. The following vehicles have to slow to avoid the right-turn vehicle. The second and third through vehicles also slow and follow at the minimum headway $h$ to the through vehicle ahead. The delay $d$ incurred by each through vehicle is computed as the time lag in its trajectory.

**EXHIBIT A15-7.** Time-space representation of through and right-turn vehicles.
Performance Measures Module

The Performance Measures module is used to compute the delay and stop rate due to random arrivals. This delay and stop rate are combined with the deterministic delay and stop rate computed in the Queue Accumulation Polygon module to obtain a total delay and stop rate for the segment. These measures are then used to compute the segment travel speed and spatial stop rate.

This section describes three routines that are used to quantify the delay due to turns at an access point. These routines are:

- Incremental Delay Routine
- Overflow Stop Rate Routine
- Travel Speed and Spatial Stop Rate Routine

A brief overview of each routine is described in the following subsections.

Incremental Delay Routine

The Incremental Delay routine is used to compute the incremental delay for each intersection lane group. Equation 15-12 is used for this purpose. As noted in the discussion associated with this equation, the capacity used for the non-coordinated phases is the “available capacity.” This capacity is based on the equivalent maximum green duration and reflects the ability of an actuated phase to minimize delays due to random arrivals.

Overflow Stop Rate Routine

The Overflow Stop Rate routine is used to compute the incremental delay for each intersection movement. Equation 15-20 is used for this purpose. As noted in the discussion associated with this equation, the capacity used for the non-coordinated phases is the “available capacity.” This capacity is based on the equivalent maximum green duration and reflects the ability of an actuated phase to minimize stops due to random arrivals.

Equation 15-20 is based on Equation G16-9 in Chapter 16. However, it has been modified to estimate the overflow queue in the lane group, as opposed to an individual lane. The modification entails the use of lane group capacity in Equation 15-20, as opposed to lane capacity.

Travel Speed and Spatial Stop Rate Routine

The Travel Speed and Spatial Stop Rate routine is used to compute the travel speed and spatial stop rate for the segment and the system. Equation 15-25 is used to compute the segment travel speed. Equation 15-26 is used to compute the system travel speed. Similarly, Equation 15-27 is used to compute the spatial stop rate for the segment and Equation 15-28 is used to compute the spatial stop rate for the system.
APPENDIX B. FIELD STUDY PROCEDURE

The following steps can be used to determine the base free-flow speed for an urban street segment.

1. Conduct a spot-speed study at a typical mid-segment location during low-volume conditions. Record the speed of 100 or more free-flowing passenger cars. A car is free-flowing when it has a headway of 8 s or more to the vehicle ahead and 5 s or more to the vehicle behind in the same traffic lane.

2. Compute the average of the spot speeds, $S_{\text{spot}}$, and their standard deviation $\sigma_{\text{spot}}$.

3. Compute the average segment free-flow speed $S_f$ as a space-mean speed using the following equation:

   $S_f = S_{\text{spot}} - \frac{\sigma_{\text{spot}}^2}{S_{\text{spot}}}$

   where,
   
   $S_f = $ free-flow speed, mph.

4. Compute the base free-flow speed $S_{fo}$ using the following equation:

   $S_{fo} = \frac{S_f}{f_L}$

   with,

   $f_L = 1.02 - 4.7 \frac{S_f - 19.5}{L} \leq 1.0$

   where,
   
   $S_{fo} = $ base free-flow speed, mph;
   
   $L = $ segment length, ft; and
   
   $f_L = $ segment length adjustment factor.

Equation B15-3 was originally derived to use the base free-flow speed $S_{fo}$ in the numerator of its second term. However, it is sufficient for this application to substitute the free-flow speed $S_f$. If segment length is less than 400 ft, then it should be set to 400 ft for use in Equation B15-3.

The following steps can be used to determine the average travel speed for an urban street segment.

1. Identify the time of the day (e.g., morning peak, evening peak, off-peak, etc.) during which the study will be conducted. Identify the segments to be evaluated.

2. Conduct the test-car travel time study for the identified segments during the identified study period. The following factors should be considered before, or during, the field study.
   a. The number of travel-time runs to be conducted will depend on the range of speeds found on the street. Six to twelve runs for each traffic-volume condition are typically adequate. The analyst should determine the minimum number of runs based on guidance provided in Chapter 4 of Reference 7.
   b. The objective of the data collection is to obtain the information identified in the Travel Time Field Worksheet (i.e., vehicle location and arrival/departure time at each boundary intersection). In general, each row of this worksheet represents the data for one direction of travel on one segment. If the street serves traffic in two travel directions, separate worksheets are typically used to record the data for each direction of travel.
c. The equipment used to record the data may include a GPS-equipped laptop computer or simply a pair of stopwatches. If available, an instrumented test car should be used to reduce labor requirements and to facilitate recording and analysis.

d. During the test run, the average-car technique is typically used and requires that the test car travel at the average speed of the traffic stream, as judged by its driver.

e. The cumulative travel time is recorded as the vehicle passes the center of each boundary intersection. Whenever the test car stops or slows (i.e., 5 mph or less), the observer uses a second stopwatch to measure the duration of time the vehicle is stopped or slowed. This duration (and the cause of the delay) is recorded on the worksheet on the same row that is associated with the next boundary intersection to be reached. The rows are intentionally tall so that a mid-segment delay and the signal delay can both be recorded in the same cell.

f. Test-car runs should begin at different time points in the signal cycle to avoid having all runs start from a “first in platoon” position.

g. Some mid-segment speedometer readings should also be recorded to check on unimpeded travel speeds and to see how they relate to the estimated free-flow speed.

3. The cumulative travel time between adjacent boundary intersections can be subtracted to obtain the travel time for the corresponding segment. This travel time can then be averaged for all test runs to obtain an average segment travel time. This average is then divided into the segment length to obtain an estimate of the average travel speed. This speed should be computed for each direction of travel for the segment.

   The data should be summarized to provide the following statistics for each segment and travel direction: average travel speed, average delay time for the boundary intersection, and average delay time for other sources (pedestrian, parking maneuver, etc.).

   The average segment travel time for a common direction of travel should be added to obtain the total travel time for the facility. This total travel time can then be divided into the facility length (i.e., the total length of all segments), to obtain the average travel speed for the facility. This calculation should be repeated to obtain the average travel speed for the other direction of travel.
APPENDIX C. WORKSHEETS

Running Time Worksheet
Proportion Arriving During Green Worksheet
Control Delay Worksheet
Stop Rate Worksheet
Travel Speed and Spatial Stop Rate Worksheet
Travel-Time Field Worksheet
## Running Time Worksheet

### General Information
- Analyst
- Agency or Company
- Date Performed
- Analysis Time Period
- Street
- Jurisdiction
- Analysis year
- Level of Analysis

### Site Information

### Input Data

<table>
<thead>
<tr>
<th>Segment</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td><strong>Direction of Travel</strong></td>
<td>EB/NB</td>
<td>WB/SB</td>
<td>EB/NB</td>
<td>WB/SB</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Segment Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of through lanes for length of segment (N), ln</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Speed limit (Spl), mph</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mid-segment volume (v), veh/h</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total delay due to turns into access points (Σdturn), s/veh</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Delay due to other mid-segment sources (dother), s/veh</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length of segment (L), ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Width of upstream boundary intersection (w), ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length of segment with restrictive median (Lrm), ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length of segment with non-restrictive median (Lnr), ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Start-up lost time (l1), s</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Access Data

| | | | | |
|---|---|---|---|
| Percent of street with curb on right-hand side (Pcurb), % | | | |
| Number of access points on right-hand side (Nap) | | | |

### Running Time Computation

\[ L_{adj} = L - w \]

\[ P_m = \frac{100 \times L_{rm}}{L_{adj}} \%

\[ S_0 = 25.6 + 0.47 \times S_{pl} \]

\[ f_{CS} = 0.015 \times P_m - 0.0047 \times P_{curb} + 0.0037 \times P_{curb} \times P_m \]

\[ D_a = \frac{5280 \times (N_{ap,EB/NB} + N_{ap,WB/SB})}{L_{adj}} \]

\[ f_a = \frac{-0.078 \times D_a}{N} \]

\[ S_f = S_0 + f_{CS} + f_a \]

\[ f_{l1} = 1.02 - 4.7 \times \left( \frac{S_{ap} - 19.8}{L} \right) \]

\[ S_r = S_f \cdot f_{l1} \]

\[ f_{l} = \frac{2}{1 + \left( 1 - \frac{v}{5280 \times S_r} \right)^{0.31}} \]

\[ T_R = \frac{6.0 - l_1}{0.0025 \times L} + \frac{3600 \times L}{5280 \times S_r} + f_{ap} + d_{other} \]

**Note:** The first term in the running time equation is only applicable to segments with signalized or stop-controlled boundary intersections.
### PROPORTION ARRIVING DURING GREEN WORKSHEET

**General Information**

- **Project Description**

**Input Data**

<table>
<thead>
<tr>
<th>Direction of travel</th>
<th>Segment</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>EB/NB</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WB/SB</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Signal Timing Data**

- Effective green-to-cycle-length ratio ($g/C$)

**Traffic Data**

- Arrival type

**Proportion Arriving During Green Computation**

- Platoon ratio ($R_p$) \[ R_p = \frac{\text{Arrival type}}{3} \]
- Proportion arriving during green ($P$) \[ P = R_p \frac{g}{C} \]
## CONTROL DELAY WORKSHEET

### General Information

Project Description

### Input Data

**Analysis Period** \((T)\), h: ___.

<table>
<thead>
<tr>
<th>Segment</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
</table>

**Direction of travel**

<table>
<thead>
<tr>
<th></th>
<th>EB/NB</th>
<th>WB/SB</th>
<th>EB/NB</th>
<th>WB/SB</th>
</tr>
</thead>
</table>

### Signal Timing Data

**Cycle length** \((C)\), s

**Effective green-to-cycle-length ratio** \((g/C)\)

### Traffic Data

**Lane group volume** \((v)\), veh/h

**Lane group saturation flow rate** \((s)\), veh/h/ln

**Initial queue** \((Q_b)\), veh

**Proportion of arrivals during green** \((P)\)

### Geometric Design Data

**Lane group lanes** \((N)\), ln

### Delay Computation

**Available effective green time** \((g_a)\) s, Equ. 15-15 or 15-16

**Capacity** \((c)\), veh/h

\[ c = N s g / C \]

**Available capacity** \((c_a)\), veh/h

\[ c_a = N s g / C \]

**Volume-to-capacity ratio** \((X)\)

\[ X = v / c \]

**Available volume-to-capacity ratio** \((X_a)\)

\[ X_a = v / c_a \]

**Uniform delay** \((d_i)\), s/veh

\[ d_i = \frac{0.5 C (1 - g / C)^2}{1 - m h (1, X) g / C} \]

**Incremental delay calibration factor** \((k)\) (Exhibit 16-3)

**Upstream filtering adjustment factor** \((I)\)

\[ I = 1.0 - 0.91 X_a^{0.88} \]

**Incremental delay** \((d_2)\), s/veh

\[ d_2 = 900 T \left( X_a - 1 \right) + \frac{8 k / X_a}{c_s T} \sqrt{\left( X_a - 1 \right)^2 + \frac{8 k / X_a}{c_s T}} \]

**Initial queue delay** \((d_3)\), s/veh (Chapter 16, Appendix F)

**Flow ratio** \((y)\)

\[ y = m h (1, X) g / C \]

**Progression adjustment factor** \((PF)\)

\[ PF = \frac{1 - P}{(1 - g / C)^2} \left( 1 - y \right) \left( 1 - g / C + y \left( 1 - P C / g \right) \right) \]

**Control delay** \((d)\), s/veh

\[ d = d_i (PF) + d_2 + d_3 \]
## STOP RATE WORKSHEET

### General Information

<table>
<thead>
<tr>
<th>Project Description</th>
<th></th>
</tr>
</thead>
</table>

### Input Data

<table>
<thead>
<tr>
<th>Analysis Period ($T$), h</th>
<th>Segment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>EB/NB</td>
</tr>
</tbody>
</table>

### Direction of travel

<table>
<thead>
<tr>
<th>EB/NB</th>
<th>WB/SB</th>
</tr>
</thead>
</table>

### Signal Timing Data

<table>
<thead>
<tr>
<th>Cycle length ($C$), s</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective green-to-cycle-length ratio ($g/C$)</td>
<td></td>
</tr>
</tbody>
</table>

### Traffic Data

<table>
<thead>
<tr>
<th>Lane group volume ($v$), veh/h</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane group saturation flow rate ($s$), veh/h/ln</td>
<td></td>
</tr>
<tr>
<td>Initial queue ($Q_0$), veh</td>
<td></td>
</tr>
<tr>
<td>Proportion of arrivals during green ($P$)</td>
<td></td>
</tr>
<tr>
<td>Speed limit ($S_p$), mph</td>
<td></td>
</tr>
</tbody>
</table>

### Geometric Design Data

<table>
<thead>
<tr>
<th>Lane group lanes ($N$), ln</th>
<th></th>
</tr>
</thead>
</table>

### Stop Rate Computation

#### Available effective green time ($g_a$), s

Equation 15-15 or 15-16

#### Effective green time ($g$), s

$g = C \left(\frac{g}{C}\right)$

#### Effective red time ($r$), s

$r = C - g$

#### Capacity ($c$), veh/h

$c = N s g/C$

#### Available capacity ($c_a$), veh/h

$c_a = N s g_a/C$

#### Volume-to-capacity ratio ($X$)

$X = v/c$

#### Available volume-to-capacity ratio ($X_a$)

$X_a = v/c_a$

#### Average speed ($S_a$), mph

$S_a = 0.95 \left(25.8 + 0.47 S_p\right)$

#### Threshold accel.-decel. delay, s

$(1 - P)gX$

#### Acceleration-deceleration delay ($d_a$), s

$d_a = 0.393 (S_a - 5.0)^2 / S_a$

#### Deterministic stop rate ($h_1$), stops/veh

$h_1 = 1 - P(1 + d_a/g)$  

: if $d_a < (1 - P)gX$

$h_1 = \frac{(1 - P)(r - d_a)}{r - (1 - P)gX}$  

: if $d_a > (1 - P)gX$

#### Upstream filtering adjustment factor ($I$)

$I = 1.0 - 0.91 X_a^{2.88}$

#### Second-term incremental factor ($k_b$)

$k_b = 0.12 I \left(\frac{g}{3600}\right)^{0.7}$  

(use Equ. 15-22 for non-coord.)

#### Overflow queue ($Q_o$), veh

$Q_o = 0.25 c_a T \left[\left(X_s - 1 + \frac{Q_o}{c_s T}\right)^2 + \left(\frac{X_s - 1 + Q_o}{c_s T}\right)^2 + \frac{8 k_b X_s}{c_s T} + \frac{16 k_b Q_o}{(c_s T)^2}\right]$

#### Full stop rate ($h$), stops/veh

$h = h_1 + \frac{Q_o}{XV_C}$
### General Information

Project Description

### Input Data

<table>
<thead>
<tr>
<th>Segment</th>
<th>System Total ¹</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Direction of travel</th>
<th>EB/NB</th>
<th>WB/SB</th>
<th>EB/NB</th>
<th>WB/SB</th>
<th>EB/NB</th>
<th>WB/SB</th>
</tr>
</thead>
<tbody>
<tr>
<td>EB/NB</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WB/SB</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Length of segment (L), ft |       |
| Base free-flow speed (S₀), mph |       |
| Running time (Tᵣ), s |       |
| Control delay (d), s/veh |       |
| Full stop rate (h), stops/veh |       |
| Full stop rate due to other mid-segment sources (hₜₙ), stops/veh |       |

### Travel Speed Computation

- **Travel time (Tₜ), s**
  \[ Tₜ = Tᵣ + d \]

- **Travel speed (Sₜ), mph**
  \[ Sₜ = \frac{3600 \times L}{5280 \times (Tᵣ + d)} \]

### Spatial Stop Rate Computation

- **Total stop rate (hₜ), stops/veh**
  \[ hₜ = h + hₜₙ \]

- **Spatial stop rate (H), stops/mi**
  \[ H = \frac{5280 \times (h + hₜₙ)}{L} \]

### Level of Service Computation

- **Travel speed as a percentage of base free-flow speed**
  \[ R = 100 \times \frac{Sₜ}{S₀} \]

### Note:

1 - Calculation code: s - segment sum; a - segment average (see text); e - use equation in column 1.

---

Chapter 15 - Urban Streets

15-80
<table>
<thead>
<tr>
<th>Location (typically a boundary intersection)</th>
<th>Run Number: ______________________________</th>
<th>Run Number: ______________________________</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cumulative Travel Time at Location (s)</td>
<td>Delays Due to Slow or Stop</td>
<td>Cumulative Travel Time at Location (s)</td>
</tr>
<tr>
<td>Cause 1</td>
<td>Delay Time (s)</td>
<td>Cause 1</td>
</tr>
</tbody>
</table>

1 - Cause of delay: Ts - signal; Lt - left turn; Pd - pedestrian; Pk - parking; Ss - stop sign; Ys - yield sign.